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MEMBERSHIP

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PAPERS

EFFECTS OF BENDING WIRE ROPE

By Frederick C. Carstarphen[1], M. Am. Soc. C. E.

Synopsis

An ever-present suspicion appears in the discussion of wire rope to the effect that the popular formulas for the estimation of the loss of strength due to bending may not be consistent with experiment. This can be settled only by holding the problem up to engineering and analytical scrutiny.

This paper is a start along this path. New and more comprehensive formulas are presented, which may serve a useful purpose in bringing to light data and analyses of the problem that have not been published by those who have given their attention to the subject. Wire rope is one of the most used, and most abused, commodities of industry. Therefore, all its properties should be known, to the end that the public and the wire rope manufacturer may agree upon the increase in its cost, if necessary to provide for more safeguards in the selection of materials and in the process of manufacture, so that its performance may be predicted for a properly supervised service.

***Notation.*—The symbols used in this paper are defined at the place in which they are first introduced in the text and, in addition, they are completely listed and defined in Appendix I.**

Historical Notes

The name of the inventor of wire rope is probably lost forever in the mist of antiquity. Samples of wire rope made of bronze have been excavated among the ruins of Pompeii; but, apparently, they consist of three strands twisted without the use of a center. Albert of Clausthal made ropes of both regular and lang lay in 1834, and for some years thereafter, and discussed haulage technique.[2] In America, credit is given to the late John A. Roebling, M. Am. Soc. C. E., as the pioneer wire rope maker. He manufactured a

Note.—Written discussion on this paper will be closed in March, 1932, *Proceedings*.

[1] Cons. Engr., Denver, Colo.

[2] *Mining Journal*, London, England, 1837.

piece of $\frac{3}{4}$-in., 6 × 7, iron rope, about 300 ft. long, at Saxonburg, Pa., in 1840. This rope, which was used on the canal plane near Portage, Pa., was made on a ropewalk such as was formerly used in the manufacture of fiber ropes. In 1848, the works were moved to Trenton, N. J., and wire ropes were made for use on suspension bridges, for towing canal-boats, and for such other service as required greater strength than could be secured from hemp ropes of reasonable size.

The excellent qualities possessed by wire rope soon won for it deserved popularity and, in 1886, the Trenton Iron Company undertook its manufacture, using machines designed in accordance with the best English practice. As in the case of many other innovations, the market was soon "saturated," and in order to stimulate sales each manufacturer began to offer different types, styles, and grades of rope. This was the period when iron was being abandoned for steel, and it was the proper thing to claim that certain ropes were made of the highest grade crucible cast steel, plow steel, and super-plow steel. It was not sufficient alone to vary the grades of steel, but the construction of wire rope was varied so as to appeal to the whim of the most exacting customer, and each change was always heralded as being a superior product and claims were made that it could not be excelled for the use intended. Although many bizarre constructions of rope have been abandoned, many wire rope catalogs contain more than eighty tables giving the properties of the different styles of wire rope that are manufactured. To quote Mr. Thomas Woodhouse, of Dundee, Scotland:[3]

"The introduction of aerial ropeways as a means of transportation some 50 or 60 years ago, created a demand for steel wire ropes previously non-existent. * * * Aerial ropeways now provide a big outlet for steel wire ropes of almost all constructions."

Terminology

It is reasonable to assume that with so many different kinds of wire rope, confusion sometimes arises concerning its properties and its ability to render service. As a matter of fact, wire rope is nothing more than an assemblage of wires arranged so as to permit it to meet, in the best manner, several general conditions which bound the limit of its use. It may be ideally adapted to discharge a certain duty, but as the limits of its usefulness are extended the border line is reached at which failure is encountered, not through any inherent defect in the wire rope, but because it is not capable of service in an infinite number of fields. There is no other material used for engineering purposes which will stand greater abuse and still function satisfactorily than wire rope; yet it has its limitations and should be used accordingly. It is proposed, therefore, to consider several properties of wire rope which follow from the nature of its construction.

In the remarks that follow, the lines that are of such size and construction that they can be put in motion, will be considered as "ropes," and those that remain at rest, as "cables." The term, "cable," is also applied to ropes of

[3] Encyclopedia Britannica, Edition XIV, Vol. 23, p. 675.

large size, as is also the word, "hawser," when the cable is used for towing purposes. In the electrical field, the words, "cables" and "guys," are in general use.

Wire rope is an assemblage of wires, and the strength of each wire depends upon the composition of the material from which it is drawn, the number of drafts given, the diameter, and the heat treatment which it receives. Hence, it follows that those wires which have been subjected to the greatest number of drafts, and are properly heat-treated, have a higher tensile strength than those of larger size. Unfortunately, wires of small diameter will not stand abrasion and abuse without breaking. It is concluded, therefore, that strong ropes should be built of a number of small wires. Those which are subject to surface wear should be built of larger wires. When both strength and resistance to wear are desired these two properties must be combined, and it is here that the skill of the rope maker is best shown.

Obviously, as long as ropes remain stationary and sustain loads which do nothing more than cause a variation in tension, it is quite sufficient to gather the wires together in a bundle and serve them with wire or tape so as to preserve their grouping and protect them from the weather. Cables of this so-called "selvagee" construction are used on suspension bridges, particularly because this type of cable may be erected in position by stringing the individual wires in place between the bridge towers. To speed the completion of suspension bridge cables, wires are assembled in groups of 200 or more, or as wire cables, and are set into place to make the bridge cable, thus doing in days what formerly required months by "spinning."

Characteristics

In addition to strength, wire rope must acquire two other qualities, namely, flexibility and resistance to wear. Since fiber ropes have been made from the earliest times, it is natural to wonder why the development of wire rope did not advance accordingly; but like all great innovations, apparently simple, it was impossible until several associated problems were solved. Just as it is impossible to build an arch without a center, so it was impossible to undertake the construction of wire rope on a commercial scale until sufficient progress had been made in the wire and steel industries, that would insure the production of rods of such quality and quantity that they could be drawn into long wires possessing adequate strength. Prior to about 1850 the iron and steel industry had not made sufficient progress to permit the development of wire rope as it is now known.

Some time in the early part of the Nineteenth Century the important discovery was made, by either Albert of Clausthal, or J. Wilson of Derby, or by both independently, that flexibility could be secured by using fine wires twisted into strands and that the strands could be laid about a fiber center to form ropes. As this process can be done in a great variety of ways, different manufacturers adopted different designs for the symmetrical arrangement of the wires in the strands, the direction of the twist, and its pitch, which is called the lay. Reference to any company's rope list will show the

arrangements which are considered to be standard. As experience was gathered in the use of wire rope there has been a gradual shortening of the lay of the wires and strand. In 1890 it was not uncommon to find 6 × 7 wire rope having a lay of wires forming the strand equal to thirty times the diameter of the wire, and a lay of rope equal to nine times the diameter of the strand. In modern practice the corresponding ratios would be nearly 22.5 and 6.5. In other words, with increased tensile strength of wires, it is possible to increase flexibility without a serious sacrifice of strength, although the tables adopted by the Wire Users Association in 1910 and in 1929 each assigned lower values to the breaking strengths of plow and super-steel ropes than formerly.

As an aid in estimating the theoretical number of wires in the diameter of a rope, the lay of the strand, and the lay of the rope, in terms of the diameter of the wires, Table 1 is presented.

TABLE 1.—STANDARD CHARACTERISTICS OF WIRE ROPE

Kind of rope	Lay of strand*	Lay of rope*	Number of wires in the diameter of rope
7-wire, ordinary	22.4d′	58.3d′	9
7-wire, lang lay	22.4d′	62.8d′	9
Seale, ordinary	29.4d′	76.6d′	12
Seale, lang lay	31.4d′	82.6d′	12
19-wire, ordinary	35.7d′	93.0d′	15
19-wire, lang lay	38.1d′	100.0d′	15

* d′ = diameter of wire.

The strengths of the various steels used in the manufacture of wire rope (in kips[4] per square inch), are generally accepted, as follows:

Iron wire	85 to 105
Mild cast steel	160 to 180
Cast steel	180 to 200
Extra cast steel	200 to 220
Plow	220 to 240
Super-plow	240 to 280

All the foregoing varieties of steel are used in the manufacture of wire rope. However, it is not sufficient to examine the wires for tensile strength alone; they must also be examined for torsion, bending, and elongation. The tensile strength and elongation are determined in the usual manner in a standard testing machine. When elongations are required an extensometer is used to ascertain the increments in loading. When the yield point is reached the instrument is removed. From the data secured, plus a knowledge of the area and the length of the specimen tested, it is possible to calculate the modulus of elasticity of the steel. This factor varies, of course, depending upon the composition of the steel, but will usually be found to be between the limits of 25 000 to 30 000 kips per sq. in. In other words, the modulus of elasticity is the load required (theoretically) to extend the specimen a

[4] 1 kip = 1 000 lb.

distance equal to its own length, and is of considerable importance in solving problems connected with the stresses caused by changes in length of the specimen, whether it be by temperature, or by exterior or interior loading.

The torsion test is made to determine the number of twists about a longitudinal axis which a wire will stand and, in a way, it is a measure of its ductility. Wires possessing a diameter greater than 0.04 in. are mounted in a machine constructed so that it has two opposed heads fitted with chucks or clamps for holding the wire. These heads are usually spaced 8 in. apart and run 33 rev. per min. in opposite directions, giving an equivalent twisting a speed of 66 rev. per min. Means are provided for placing the wire under a known tension, thus standardizing the test. The torsion test quickly reveals brittle wires and is confirmed by the results of the kink and jig tests. Wires which are sufficiently small to be bent by hand into loops such that the diameter is approximately that of the wire, are tested accordingly. This is the kink test and is usually confined to wires less than 0.04 in. in diameter. Larger wires are bent 120° in four directions about dies of a given radius, depending upon their size, by means of a special device called a jig. As an illustration of the variation of the torsion and bending resistance in different grades, Table 2 is presented for a 0.111-in. wire such as would be used in a 1-in., 6 × 7 rope. The torsion specimens are under a tension of 23 lb.

TABLE 2.—Torsion Test of Wires, 0.111 Inch in Diameter

Grade	Breaking strength, in kips	Twists in 8 in.	Jig test (in standard jig)	Percentage elongation
Mild steel	1.57	26	Four 120° bends	2.3 to 3
Cast steel	1.70	26	Four 120° bends	2.3 to 3
Extra strong cast steel	1.96	25	Four 120° bends	1.8 to 2.8
Plow	2.16	24	Four 120° bends	1.8 to 2.6
Super-plow	2.36	22	Four 120° bends	1.6 to 2.6

It is possible to make wire for ropes from both basic and acid steels up to the plow grade, but open-hearth acid steels are preferred. It is understood that steel wire possessing the requisite properties for making suitable wire rope can be secured only by a careful control of the composition of the steel in the furnace, by a proper cropping of blooms, and by the discreet rolling of billets into rods. These steps must be followed by the elaborate and painstaking processes of cleaning, baking, wire-drawing, annealing, and heat treating, or "patenting," all of which combine to make possible the production of the high tensile strengths which characterize wires used for this purpose.

Table 3 illustrates the progressive increase in strength acquired by an acid open-hearth steel rod, $\frac{5}{16}$ in. in diameter, due to wire-drawing. The specimen had the following composition: Carbon, 0.8%; manganese, 0.6%; silica, 0.14%; sulfur, 0.009%; phosphorus, . . .; and copper, 0.03 per cent. The fact that the elongation in 10 in. had dropped from 4% to less than 1.1% emphasizes the interdependence between the elasticity and strength. In other words, flexibility is not a property of wires possessing high resistance to

rupture. Accordingly, wires possessing a strength of 300 kips per sq. in. are usually so brittle as to be considered worthless for wire rope manufacture. This statement is qualified for steels of music-wire grade, and a few alloyed steels, the cost of which precludes them from use in wire ropes.

TABLE 3.—INCREASE IN STRENGTH ACQUIRED BY WIRE DRAWING

Observation	Diameter, in inches	Breaking Strength		Percentage elongation, in 10 in.
		In kips	In kips per square inch	
0	0.3125	4.60	60	(Unstressed)
1	0.191	5.160	180	4.0
2	0.159	3.975	200	3.5
3	0.132	3.145	230	2.4
4	0.093	2.095	308	1.1

MECHANICAL DEVELOPMENT OF MANUFACTURE

When the testing room passes the wire as satisfactory, it is delivered to the rope shop and wound upon bobbins for use in the strand-making machines. The development of wire-rope stranding machines is an interesting one. When Mr. Roebling undertook the manufacture of his first wire rope it was done in a rope walk, similar to the method used in the manufacture of fiber ropes. Albert of Clausthal, the accredited inventor of wire rope, followed a similar method of manufacture. It happens that, in 1838, Mr. R. S. Newall, of Dundee, Scotland, received a letter from a friend commenting upon the method in vogue in Saxony for making wire rope, consisting of four strands of four wires, $\frac{3}{32}$ in. in diameter, twisted together. The friend stated that the process was exceedingly crude and urged Mr. Newall to design a machine for this purpose. Within a month he had prepared the plans of the machine, and, in August, 1840, secured the British patents for the production of wire rope formed by closing strands about a core.

For many years after the introduction of this machine it was felt that means must be used to prevent twisting the individual wires when incorporated into the strand. This resulted in keeping the velocity of the machine at a minimum, 40 to 50 rev. per min. being considered a maximum. An American, Mr. J. B. Stone, of Worcester, Mass., invented a stranding machine which eliminated the gears and other equipment used to prevent torsion in the wires, and, as a result, higher operating speeds were secured.

WIRE-STRANDING MACHINES

The type of machine most favored in America consists of a suitable bedplate, supporting rollers; these, in turn, carry the flyers which are connected by rods. The flyers are placed so as to accommodate a floating cradle which, in turn, carries a bobbin. On the 19-wire stranding machine provision is made for supporting nineteen bobbins. The cradle is constructed with a counterweight and is mounted in the machine so that it does not revolve

when the machine is in motion. By giving over to gravity the control of the position of the bobbins, the machine may be operated at high speed. The wires are conducted to the front end of the machine and are led from the ports of the flyers through the slots of the twisting head which wraps them about the wire center.

There must also be longitudinal motion to form the strand. The machine is equipped, therefore, with a draw-out mechanism, which consists of multiple grooved sheaves arranged so as to revolve at a speed which bears a fixed ratio to the number of revolutions of the machine proper. As an illustration, for a certain size of strand the draw-out mechanism handles 1 in. of strand per revolution of the machine. If the draw-out mechanism should cease to function, the strand would soon be twisted in two. In order to vary the velocity of the draw-out mechanism a speed-changing device is used. For instance, two spur gears may be carried in a frame equipped with a handle and mounted upon a spline shaft. When the operator wishes to change the speed ratio it is only necessary to raise the frame, slide it along the shaft until the proper driving gear is encountered, and then drop it into position. Since right or left-lay strands may be required, it is necessary for the draw-out mechanism always to run in the same direction irrespective of the rotation of the machine. This is accomplished by using opposed bevel gears and a pinion.

These machines are rated according to the weight of the wire held by the bobbins, such as 50, 100, or 250-lb. machines. The greater the weight of the bobbin the larger the machine and the slower it must run because of the rotative forces developed. This machine is not rigid, and each flyer is supported on rollers. Both ends of the machine are driven from a countershaft which extends its full length. In practice, it is found difficult to keep a constant pressure on the several rollers. The machine is also equipped with multiple solenoid brakes that act simultaneously and tend to reduce unbalanced stresses in the frame when being brought to rest.

Machines have been built so that strands composed of more than nineteen wires could be laid up in a sequence of continuous operation. Such a machine would be called a tandem compound wire-stranding machine. The first unit would be arranged so as to lay the six wires on the center, the next would place twelve wires, then eighteen, and finally, twenty-four, making a strand of sixty-one wires. Unfortunately, the speed of the assembly depends upon the rate of laying the initial six wires. Since these tandem machines were of considerable length, it was necessary to group each multiple of six flyers in the same transverse plane, giving the planetary arrangement. This arrangement entailed flyers of large dimensions, so that the centrifugal stresses were considerable. The result secured was not satisfactory because of the lack of speed of the assembly. It has been found preferable to prepare the strands as a series of single operations and thereby increase the speed of each one to the limit of economical and satisfactory manufacture. Compound rope machines were also perfected based on the idea of arranging six stranding machines so as to manufacture the strand and close the rope in

one continuous operation. These machines have also been superseded because of their inability to give a satisfactory output in competition with superior high-speed stranding machines and a closing machine which run independently at efficient speeds.

The method of assembling the strands into wire rope is very similar to that used in making strand, except that the rope is formed by closing the strands about the fiber or wire rope center. The speed of a closing machine is much less than that of a stranding machine, because of the great increase in weight of the revolving parts. A fair speed is 45 rev. per min.

Types of Wire Rope

There are several other classes of cable which are worthy of attention. Locked-coil cables consist of a center of seven or nineteen round wires. When of sufficient diameter the core is covered with several layers of trapezoidal or key wires. The exterior surface consists of a layer of interlocking wires which are placed so snugly as to give a smooth and cylindrical appearance to the cable. Unfortunately, these wires are of large relative cross-section, and as made in America, do not exceed 150 kips per sq. in. in tensile strength (secured by special heat treatment called "patenting").

Half-lock cables are wire strands covered with round and shaped wires to give a smooth exterior. Wires of relatively large diameter are used. In order to improve the tensile properties of the cable, the expedient has been adopted of using many layers of small cross-sectional key wires and covering the assemblage with lock wires. This type of cable is known as locked-wire cable, the word, "wire," conveying the idea of the small key wires in its construction.

Cables formed of nineteen, thirty-seven, and sixty-one round wires as a single strand are known as track strand or smooth-coil cables. All the types aforementioned find their primary use as track cables for aerial tramways and cableways.

Experience in the use of wire rope has demonstrated that one of the most vital factors for long life is interior lubrication. Therefore, the most modern practice of rope construction demands that a center be used which is thoroughly impregnated with a lubricant. Various methods have been tried, such as running the rope, when closed, through a bath of melted petroleum or a solution of vaseline in fuel oil. The drawback to this system comes from the fact that commercial African or Javanese hemp contains considerable air and moisture which prevents the saturation of the center with the heated oil. To overcome this it is considered good practice to saturate the rope center with hot lubricant in a vacuum chamber. This process impregnates the center with sufficient oil to meet any demand which may reasonably come upon the rope, if properly cared for, during its life.

Practically all ropes of the twisted-strand construction, if more than 200 ft. in length, are wound on wooden reels, which are all built to dimensions that have been standardized by the wire companies.

Analysis of Strength

Since wire rope, from the inherent nature of its construction, is based upon the twisting of wires, it is proper to investigate the strength of the strand as compared with the sum of the strengths of the individual wires.

Example 1.—Curved Wire About a Straight Center Wire.—Fig. 1 represents a piece of straight wire, l', loaded axially by the tension, T'.

If a piece of the same kind of wire is wrapped about it so as to form a helix (Fig. 2(a)), the stress in the curved wire as compared to that in the straight one may be investigated. It is not so easy to visualize a force acting in curved wires as in straight ones, so the bent wire may be straightened

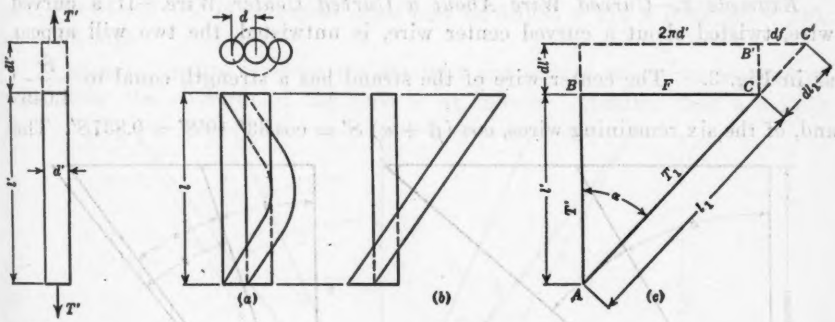

FIG. 1. FIG. 2.

after cutting both the wires by parallel planes a distance of one unit apart, which is assumed to be the lay or pitch, l', of the helix. In twisting the wire its center moves in a curve the horizontal projection of which is a circle whose diameter is equal to $2\,d'$; but the horizontal distance is equal to the circumference of this generating circle, or $2\pi d'$. The vertical distance is l'. Fig. 2(c) shows the right-angle triangle the base of which is $2\pi d'$, and the height, l'; the length of the hypotenuse, l_1, is required.

Since $l' = 22.5d'$, the hypotenuse equals,

$$\sqrt{506.25(d')^2 + 4\pi^2(d')^2} = d'\sqrt{506.25 + 39.48} = 23.36d'$$

Therefore, the length of l_1 is, $\dfrac{23.36d'}{22.5d'} \times l' = 1.03822\ l'$. Incidentally, 1.03822 is the secant of the angle of lay, which is 15° 35'.

If the two wires are considered to carry a load so that the stress in the straight wire is equal to T' (Fig. 1), and no slipping occurs between them, then the bent wire will be elongated in a direction parallel with the axis of the straight wire an equal amount, or dl'.

The triangles, ABC and $CB'C'$ (Fig. 2(c)), are similar because their sides are parallel; that is, $\dfrac{dl_1}{dl'} = \dfrac{l_1}{l'} = 1.038 = \dfrac{T_1}{T'}$. (The ratio, $\dfrac{T'}{T_1} = \dfrac{1}{1.04} = 0.96$.) The load, T_1, on the twisted wire, is $1.038T'$ If S' = the ultimate strength of the wires and is assumed to be the same for all of them,

and S'_s = the effective strength of the strand, then, $S'_s = 0.96 \times 7 \times S' = 6.72S'$, and the strand has a strength equal to 96% of the sum of the individual wires.

If this strand is to be used in a rope the lay of which is $58.3d'$, the diameter of the helix formed by the strand is equal to $6d'$, and the length of the developed strand is equal to,

$$\sqrt{6^2\pi^2(d')^2} = d'\sqrt{355.32 + 3398.89} = 61.2d'$$

Hence, in Fig. 2(*c*), $\sec \alpha = \sec \beta = \frac{61.2}{58.3} = 1.049$, or $\beta = 17°\ 35'$.

Example 2.—Curved Wire About a Curved Center Wire.—If a curved wire, twisted about a curved center wire, is untwisted, the two will appear as in Fig. 3. The center wire of the strand has a strength equal to $\frac{S'}{1.049}$, and, of the six remaining wires, $\cos(\beta + \alpha)\, S' = \cos 33°\ 10' S' = 0.837S'$. The

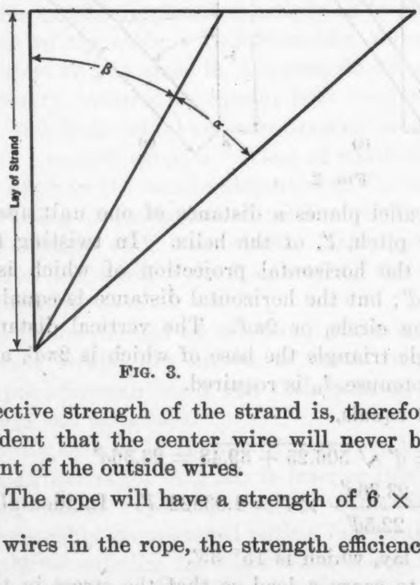

FIG. 3.

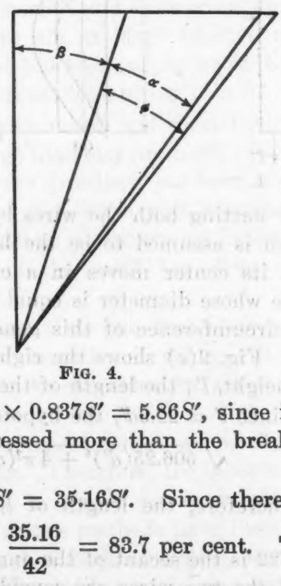

FIG. 4.

effective strength of the strand is, therefore, $7 \times 0.837S' = 5.86S'$, since it is evident that the center wire will never be stressed more than the breaking point of the outside wires.

The rope will have a strength of $6 \times 5.86S' = 35.16S'$. Since there are 42 wires in the rope, the strength efficiency is $\frac{35.16}{42} = 83.7$ per cent. This value has been established by many tests and may be compared with the rope list for 1-in., 6×7, crucible rope which is composed of 42 wires, 0.111 in. in diameter. The area of such a wire is 0.0097 sq. in., and the area of the rope is $42 \times 0.0097 = 0.4074$ sq. in.

If cast steel has an ultimate strength of 180 kips per sq. in., then the strength of the rope will be $83.7 \times 0.4074 \times 180 = 61.370$ kips, or 30.7 tons. The 1910 catalog value is 31 tons, which is a sufficient check. The value in the 1930 American Manufacturers' list is 29 tons, showing the onward decline in the standard wire strengths. Foreign rope lists carry higher breaking strengths, and should be a challenge to American rope manufacturers.

Strength of a 6 × 19 Rope.—The diameter of the wires in a 6 × 19 rope is $\frac{d}{15}$; and these are laid in a strand of 1–6–12 wires, respectively, having lays of $22.5d'$ and $35.7d'$. Let the angle of lay of the outside wires of the strand be ϕ, which is computed to be 19° 22′; the lay of the rope may be $93d'$ for modern practice, as against $112d'$ used so long in the past. To find the angle of the center of the strand, the diameter of the helix is $10d'$, and the circumference is $10\pi d'$. Then, from Fig. 5, $\sec \beta = \frac{d'}{93} \sqrt{93^2 + (10\pi)^2} = \frac{98.16}{93}$ $= 1.0554$, and $\beta = 18° \; 39'$. Then, $\cos(\beta + \phi) = \cos 37° \; 01' = 0.7986$. The strength of the 6 × 19 cast-steel cable may be 0.7986 × 114 × 0.003525 × 180, or 57.78 kips.

If slipping occurs between the straight and curved wires the method of computing the strength of the rope is as follows: Assume that a 6 × 19

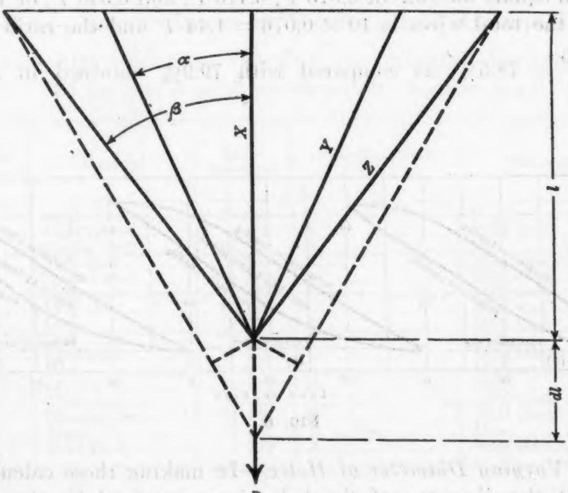

FIG. 5.

strand is arranged so that a plane projection is as shown by the solid lines in Fig. 5. The lengths of the wires are $y = X \sec \alpha$ and $z = X \sec \beta$. Then,

$$X = 6Y \cos \alpha + 12 \, z \cos \beta = P \; \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

The load on the rope is determined as follows: If $\delta X = \delta l$, then $\delta Y = \delta X \cos \alpha$, and $\delta Z = \delta X \cos \beta$, from which the following elongations may be written:

$$e_X = \frac{\delta l}{l}; \; \sigma_X = \frac{E \, \delta X}{l};$$

$$e_Y = \frac{\delta X \cos^2 \alpha}{l}; \; \sigma_Y = \frac{E \, \delta X \cos^2 \alpha}{l};$$

$$e_Z = \frac{\delta X \cos^2 \beta}{l}; \text{ and } \sigma_Z = \frac{E \, \delta X \cos^2 \beta}{l}.$$

Assuming that all wires have equal areas:

$$X = \sigma_X;\ Y = \sigma_X \cos^2 \alpha;\ \text{and}\ Z = \sigma_X \cos^2 \beta.$$

Substituting these values in Equation (1):

$$X = \frac{P}{1 + 6 \cos^3 \alpha + 12 \cos^3 \beta} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

For the rope in Fig. 6 let $\alpha = 17°\ 39'$ and $\beta = 33°\ 10'$. Substituting in Equation (2),

$$X = \frac{1}{13.2}\ P = 0.076\ P;\ 6\,Y = 6\,X\ \cos^2 \alpha = 6 \times 0.076\ P_X \cos^2 \alpha = 0.410\,P$$

and,

$$12\,Z = 12 \times 0.076\ P_X \cos \beta = 0.640\,P$$

The strength equals the sum of 0.076 P, 0.410 P, and 0.640 P, or 1.126 P. The strength of the total wires is $19 \times 0.076 = 1.44\ P$ and the ratio of strengths equals $\frac{1.126}{1.44} = 78.5\%$, as compared with 79.9% obtained in the previous calculation.

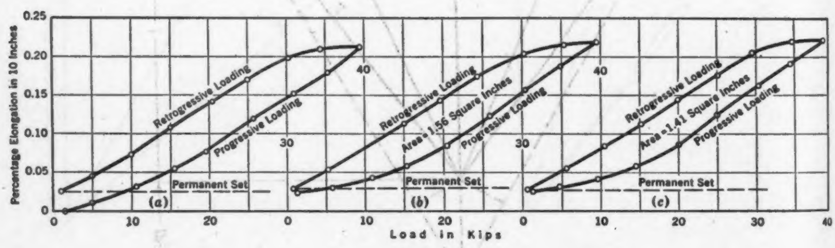

FIG. 6.

Effect of Varying Diameter of Helix.—In making these calculations it is assumed that the diameter of the helix is maintained by the core. It is quite remarkable what profound changes take place in the value of the modulus of elasticity of a rope if the helix diameter varies. Assume the value of E for the steel of the 6×19 rope as 27 000 kips per sq. in. The modulus for the rope might be $0.7986 \times 27\,000$, or 21 500 kips per sq. in. By test it may be found to be 10 750 kips per sq. in., the elongations in the latter case being twice the former. If a load of 10 kips is applied to the rope, it is easy to show that a reduction in diameter of the core of 0.007 in. is sufficient to account for the difference in the theoretical and actual moduli of elasticity. Thus, if the lay of the rope is $93d'$, and $d' = 0.067$ in., the lay is 6.231 in.; for 10 kips, the actual elongation (permanent stretch eliminated) equals $\frac{10 \times 6.231}{10\,750 \times 0.4019} = 0.0144$ in. For 21 500 kips, the amount is 0.0072 in. Theoretically, it should elongate to $6.231 + 0.0072 = 6.2382$ in., but actually

it elongates to $6.231 + 0.0144 = 6.2454$ in. Because of the modulus of the steel the length of the strand is $6.2382 \times 1.0554 = 6.5838$ in., but it is a question as to how much too short this length is to accommodate the change in length of rope from 6.231 to 6.2454 in. The answer is found thus: $(6.5838)^2 = (6.2454)^2 + C^2$; $C = 2.083$ in.

In this computation, C is the circumference of the helix circle; its radius is 0.3315 in. compared with $5 \times 0.067 = 0.3350$ in. at the beginning. The difference in the diameters of the core to meet the conditions imposed is $2 \times 0.3350 - 0.3315 = 0.007$ in. Of course, this calculation also illustrates the profound easing of the tension in a strand of a bent rope if the core yields ever so little. From experience it is known that the rope centers vary in diameter, and have varying resistances to crushing, through such wide limits that the behavior of rope in service is not a matter of exact prophecy. As an illustration of this fact scan the records of the ropes in Table 4.

TABLE 4.—SERVICE RECORD OF WIRE ROPES $\frac{5}{8}$ INCH IN DIAMETER.

(Tension, 1.25 kips; velocity, 550 feet per minute; diameter of sheaves, 34 inches; 6×19 construction; regular lay.)

Manufactured by:	Material	Miles traveled	Manufactured by:	Material	Miles traveled
A	Steel	10 247	B	Iron	10 372
C	Swedish iron	7 168	A	Mild steel	42 568
A	Special steel	18 343	B	Iron	26 130
A	Mild steel	9 158	A	Mild steel	13 538
B	Special steel	5 732	A	Mild steel	19 345
B	Mild steel	13 678	D	Mild steel	35 565
A	Mild steel	13 192	D	Mild steel	2 625
B	Iron	12 019	A	Mild steel	48 091
B	Iron	10 184	B	Iron	22 787
A	Mild steel	17 209	C	Mild steel	5 925
B	Iron	11 247	A	Mild steel	18 279
A	Mild steel	19 991	B	Iron	11 452
B	Mild steel	6 963	A	Mild steel	5 723
C	Swedish iron	24 653	B	Mild steel	12 733
B	Iron	13 487	B	Iron	3 508
B	Mild steel	4 714	A	Mild steel	8 115

PROPERTIES OF WIRE ROPE

Under tension, wire rope behaves very differently from a homogeneous bar. For instance, a sample of rope may be placed in a testing machine and subjected to stress. If an extensometer is used the strain corresponding to any stress may be noted. If the loading is progressive to a value within the yield point and is then reversed, the stress-strain diagram will fail to close by an amount equal to the permanent stretch. However, if these reversals of stress are repeated a sufficient number of times the stress-strain diagram will become a closed figure, showing that the specimen has reached a condition in which it has completely recovered from the effects of loading. The stress-strain graphs of the progressive and retrogressive loadings do not coincide, but they bound an area which represents work in a manner similar to the well-known indicator diagram. These stress-strain graphs are shown in Fig. 6.

By a proper determination of the average tension and elongation, as shown by the diagram, the amount of work lost in stretching the rope may be estimated. This result indicates the value of the internal friction developed in the interior of the rope and such a stress-strain curve is called a frictional hysteresis diagram. It will be noted that when the loading is relatively light the elongations for a constant increment of loading are not equal and the graph is curved. When the loading becomes relatively high, the stress-strain graph approaches a straight line. The explanation is simple. The modulus of elasticity of wire rope is variable under moderate tensions, but it follows Hooke's law when the loads applied develop internal friction sufficient to compare with the cohesive forces that exist in a solid homogeneous bar. It is evident from the nature of the construction of wire rope that it is not adapted to resist longitudinal compression. Therefore, the modulus of elasticity in compression lengthwise is too small to consider. The neutral axis of a bent rope seldom coincides with the gravity axis of its section.

Bending Stress in Wire Rope

The question of bending stresses in wire rope is a mooted subject. It is the usual procedure to rely upon the theory of flexure as developed for homogeneous prisms as an aid in determining these bending stresses. It is admitted by those who have a proper knowledge of the theory of flexure that it is only approximate and its use is permitted in engineering calculations because a great number of physical tests have demonstrated its limitations. It is founded on nine assumptions, three of the most important of which are:

1.—The bending must be slight;
2.—Plane surfaces before, must remain plane surfaces after, bending; and
3.—The modulus of elasticity in tension must be equal to the modulus of elasticity in compression.

In bending wire rope around the usual sheaves these criteria are violated and the use of the accredited formulas yields delusive results. Those who have studied the elastic curve of loaded beams will recall that the theory of flexure gives an approximate formula which may be written, $EI\frac{d^2y}{dx^2} = M$; that is, the modulus of elasticity times the moment of inertia times the rate of curvature of the deflected beam is equal to the external moment. However, the rate of curvature is equal to the reciprocal of the radius, and the external moment (by the theory of flexure) is,

$$E \times \frac{\text{unit stress in the outermost fiber}}{\text{distance from the internal axis to outermost fiber}}$$

Hence, the unit bending stress, b, in a homogeneous circular bar that has been slightly bent, would be given by the formula,

$$b = E\frac{r'}{R} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

in which, r' is the radius of the rod, and R, the radius of bend. When Equation (3) is used to determine the stresses arising from sharp bends, the stresses indicated are of such magnitude that no one of experience would place reliance in the results obtained.

However, a wire rope is not a homogeneous rod, but an assemblage of wires, and, for that purpose, Equation (3) has been arbitrarily modified by substituting the diameter of the wire used in the construction of the rope for the rod, giving the Reuleux formula,

$$b = E \frac{d'}{D} \quad \text{.............................} (4)$$

in which, D = diameter of sheave.

Since the diameter of a wire in a 6×19 rope is one-fifteenth the diameter of the rope, and if the modulus of elasticity of steel is taken at 28 400 kips per sq. in., then,

$$b = 1\,894 \frac{d}{D} \quad \text{..........................} (5)$$

which is the well-known Rankine formula. It will be noted that the d' in the Reuleux formula indicates diameter of wire; the d in the Rankine formula is the diameter of the rope.

A large part of the literature on wire rope is devoted to the presentation of tables and diagrams of stresses due to bending, which are based directly, or with slight modification, upon these formulas. In the opinion of many this information is most deceptive and has played a prominent part in blocking the advance of the Engineering Profession to a true knowledge of the behavior of a wire rope in service.

When an engineer purchases his first piece of wire rope he is filled with fear that it will be severely injured if subjected to bends of short radius. When the "rough and ready" rigger immediately forms a bight in the end of the rope by bending it around the usual sized thimble, he is quite sure that the rigger is taking liberties with the rope that are not justified by the tables presented by wire-rope manufacturers. When the rigger proceeds to reeve a set of wire-rope blocks which have sheaves 8 or 10 in. in diameter, with a $\frac{5}{8}$-in. or $\frac{3}{4}$-in. rope, and forthwith uses the blocks to lift weights of many tons without having the rope fall in pieces, the buyer obtains a true insight into the value of the tables of bending stress and minimum sizes of sheaves as published.

Mr. James F. Howe has stated[5] that "bending stress has been confused with the effect of bending or the loss of strength due to bending, two entirely different factors." If the stress under discussion is produced by bending and a designer is trying to determine a suitable allowance to make in his calculations for its effect, the sentence quoted becomes unintelligible; but if it is meant that the tabulated stresses are fictitious, then everything becomes clear.

To put the matter to a test, a $\frac{3}{4}$-in., 6×19 rope was cut into six pieces approximately 6 ft. in length. All were fitted with sockets carefully zinced

[5] *Transactions*, Am. Soc. M. E., December, 1918, p. 17.

into position. Of the six pieces the even numbered ones were tested in direct tension and failed, respectively, at 35.00, 35.05, and 35.10 kips. The odd numbered samples were bent around a sheave 6 in. in diameter and attached to an equalizing arm, as shown in Fig. 7. The three samples broke within the limits of 63.90, 64.00, and 64.05 kips. Since the rope in this position carries double the load, it broke at 32 kips, or 3 kips less than the 35 kips

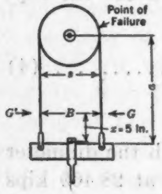

FIG. 7.

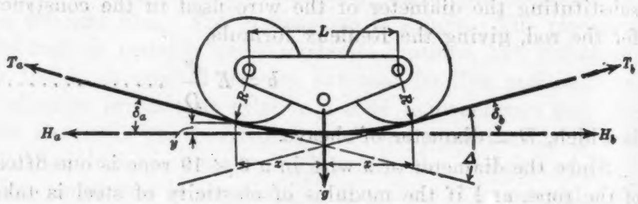

FIG. 8.

when tested in direct tension. There can be no other conclusion but that the strength of the rope was reduced 3 kips when bent 180° around a 6-in. sheave.

The area of the rope was 0.241 sq. in. and the bending stress, therefore, on the usual assumptions, was equal to 12.45 kips per sq. in. If these data are used for determining the constant for the Reuleux formula (Equation (4)), $\frac{E}{15}$ becomes $1\,285\,\frac{d'}{D}$, instead of 1 894 with $E = 28\,400$, as used in the original Equation (5), or the 12 000 as advocated by other authorities. (The answers resulting from these small constants are in kips (1 kip = 1 000 lb.).

This discrepancy between theory and practice is due to the fact that a wire rope, in passing around a sheave, has but one point of distortion, namely, the point of tangency of the rope and the sheave. This is a point of instantaneous bending, provided the rope is in motion. If the bending is performed slowly, the stress developed in the top position of a strand is distributed through its helical winding to the bottom.

The tightening of the top strands of the rope as it passes the point of contact with the sheave, causes a slight twisting of the free portion of the rope, and the resistance to this moment of torsion develops a slight curvature near the sheave. These reactions tend to reduce the magnitude of the bending stress aside from any distortion of the core. This slight rotation of the rope on guide sheaves is clearly shown by the grooving of the tread to the lay of the rope. In most cases the rotation is of the order of 0.05 in., measured on the circumference of the rope.

In testing ropes at rest on sheaves, by breaking them in tension, it has been noted that the failure often occurs in the bent portion near the point of contact of the rope and sheave. If it is admitted that bending increases the strand tension, then the point shown in Fig. 7 is the position of the greatest tension in the rope. Due to the friction of the strand contacts with the sheave in such an experiment, the tension in the rope at the crest of the

sheave is a minimum. This may be shown by a simple computation in which the tension, T, in the rope will be taken as 32 kips, the coefficient of friction, f, as 0.15, the angle of contact as $\frac{\pi}{2}$, and T_c as the crown tension,

$$T_c = Te^{-f\gamma} \qquad \text{.........................(6)}$$

or, $32e^{-0.24} = 32 \times 0.79 = 25.28$ kips.

The difference is nearly $3\frac{1}{2}$ tons. In other words, the tension in the rope along the sheave is reduced rapidly as one leaves the region of the point of contact. Therefore, at this point, the tension in the rope plus the bending effect makes it the point of maximum load and failure.

In such an experiment it is the final adjustment of the system under applied loading that determines which point of contact assumes the rôle of bending or straightening the rope. A revolving sheave determines the question at once. Just as a set of rolls bend a plate on the element of a cylinder, so does a sheave bend a rope at the point of curvature; but unlike the rolls, the sheave supports the rope in the curve developed, and then straightens it at the point of tangency. Accordingly, a sheave is a wonderful device for bending and straightening every portion of a running rope.

The reduction in the distance between the two arms of the rope loop of the test pieces, with increasing tension is noteworthy. A tension of 22.7 kips reduced the distance from $7\frac{1}{8}$ in. to $6\frac{7}{16}$ in., or $1\frac{1}{16}$ in. The amount contributed by each rope is not known, but for the sake of making a calculation, assume a span of 1.83 ft., the distance from sheave to equalizing bar, a deflection of 0.029 ft. at a point 0.25 ft. from the sheave, and compute the lateral thrust required to produce the deflection by the formula,[8] $G = \frac{yst}{x(s-x)}$. In the present case,

$$G = \frac{0.029 \times 1.83 \times 22.7}{0.25 \times 1.58} = \frac{1.205}{0.395} = 3.05 \text{ kips}$$

It may be significant that this result, derived from the maximum deflection of the system, is in almost perfect agreement with the measured reduction in strength of the rope.

A New Formula for Wire Rope

For the purposes of easy reference some of the well-known formulas for bending stress may be summarized. Rankine's formula (Equation (5)) is, $b = 1\,894\,\frac{d}{D}$. Since other formulas give total rather than unit stresses, Equation (5) is modified by introducing the area of the rope, say, $0.43\,d^2$. Then,

$$T = 1\,894 \times 0.43\,d^2 \times \frac{d}{D} = 814\,\frac{d^3}{D} \qquad \text{..............(7)}$$

[8] *Transactions*, Am. Soc. C. E., Vol. 92 (1928), p. 916, Equation (30).

For a $\frac{3}{4}$-in., 6×19 rope bent around a sheave 6 in. in diameter, Equation (7) becomes, $T = 814 \times \frac{0.75^3}{6} = 56.98$ kips, which is to be reconciled with an ultimate strength of 35 kips.

Hewitt's formula is,

$$T = \frac{E\,A}{2.06\,\frac{R}{d'} + K} \qquad \text{(8)}$$

in which, A is the cross-sectional area of a wire rope, and K, the ratio $\left(\frac{d}{d'}\right)$ of rope diameter to the diameter of the wire.

Reducing Equation (8) by inserting numerical values and substituting $2.06\ R = D$ and $d' = \frac{d}{13}$,

$$T = 910\,\frac{d^3}{D} \qquad \text{(9)}$$

Equation (9) is about the same as the Rankine formula (Equation (7)) corrected for the angle of lay of the rope.

The modified Reuleux formula (see Equation (4)) is,

$$b = E'\,\frac{d'}{D} \qquad \text{(10)}$$

in which, E' is the modulus of elasticity of the rope in tension, say, 12 000 kips per sq. in. Reducing this form to the value of:

$$T = \frac{12\,000 \times 0.43\ (d')^2 \times d'}{13 \times D} = 397\,\frac{(d')^3}{D} \qquad \text{(11)}$$

According to Equation (11), the $\frac{3}{4}$-in., 6×19 rope on a 6-in. sheave has a bending stress of 27.9 kips. In other words, the specimen will not fall to pieces before entering the testing machine.

Chapman's formula is,

$$b = \frac{E d' \cos^2 \beta \cos^2 (\alpha + \beta)}{D} \qquad \text{(12)}$$

in which, β is the angle of helix of a single wire, with the axis of the rope, and $(\alpha + \beta)$ is the angle of helix of a strand, with the axis of the rope (see Fig. 3). Bending stresses computed by Equation (12) fall between the Hewitt formula (Equation (8)), and the modified Reuleux formula (Equation (10)), and may be dismissed without further argument.

Loss of Strength in a Wire Rope Due to Bending

It has been the aim of this paper to show that wire rope is a systematic grouping of wires into helices bent about a core that may be regarded as having the properties of a yielding foundation.

Years ago it was recognized that the curving of a rod into a helix resulted in the setting up of stresses due to bending and twisting.[7] In subjecting a

[7] "Natural Philosophy", by Kelvin and Tait, Paragraph 120.

helical spring to axial tension the free end rotates, unless restrained. Accordingly, one must conclude that the wires and strands of a wire rope have been subjected to bending and twisting, and that when they are passed over a sheave, under tension, the rim of the sheave will be cut intaglio, by the rotation of the rope due to torsion. That this is so is a matter of observation, and is well known to users of wire rope. It is the manufacturer's aim to balance these effects, so that, in general, the twist of the rope may be kept at a minimum, but the torsion stresses are always present in the wires. These rim cuts raise the inquiry, "If a wire rope, under tension, passes around a sheave, is it in a state of pure bending? If not, where is the neutral surface?"

By definition pure bending is the effect produced in a bar by two equal and opposite couples acting upon its ends. In pure bending there is a gravity surface passing through the centroid of the section between the fibers in tension and compression—that is, in the region of neutrality between fibers that are elongated and those that are shortened. In general, it may be said, that as a wire rope under tension approaches the point of contact (curvature) with the sheave, all wires are under tension. When these wires pass on to the sheave surface, those in contact with it remain in tension, because the rim friction prevents the compressive accommodation. Since all wires are in tension, they are on one side of the neutral surface, which, therefore, must be at or near the rim surface of the sheave. The rim is subjected to circumferential and radial compression.

It seems that calculations for loss of strength due to bending wire rope, that are made upon the assumption of pure bending, must lead to results that are not satisfactory. For this reason there has been much speculation as to the wide discrepancy between the theoretical and the actual performance of ropes in service.

Fig. 9 shows a wire rope as an assemblage of open-coiled helical springs, made of high-class steel. It is proposed to investigate the possibility of computing rational values for the loss of strength due to bending a wire rope on this hypothesis. It is granted that the derivation of the several formulas leads to approximate answers, because it is assumed that the radius of rope of the strands, and the angle of pitch of the several wire springs remain constant, while it is evident that the former becomes less, and the latter larger, when the rope is in service. It is not believed that serious errors arise from this expedient for simplicity. It must be admitted, however, that the bending effect is a function of the tension, if precision of statement is demanded.

The mathematical theory of open-coiled helical springs is well known, and has been tested by numerous investigators, with satisfactory results. The treatment covers axial load, extension, twist, and bending. The axial bending is of importance in this problem, because all the "springs" in the rope have their axes curved about the center of rotation of the sheave.

In Fig. 9, let the heavy line represent the position of the core wire of a strand, in both a straight and a bent section of the rope. Let O be a point for investigation that may be at any point, in projection, upon the circumference of the circle of the helix. Therefore, O is defined by the radius of

the generating circle and the angle, ϕ, between this radius and the principal axial plane. The resistance offered by the open spring to axial bending results in a moment, M, which is to be used to estimate the loss in strength of the wire due to bending it. The axis of the moment, M, will be parallel to the shaft of the sheave. Let $O\text{-}P$ be a vector representing the moment, M. This vector may be resolved into two components, $O\text{-}B$ producing bending of the wire in the plane of the tangent to the surface of cylinder enclosing the

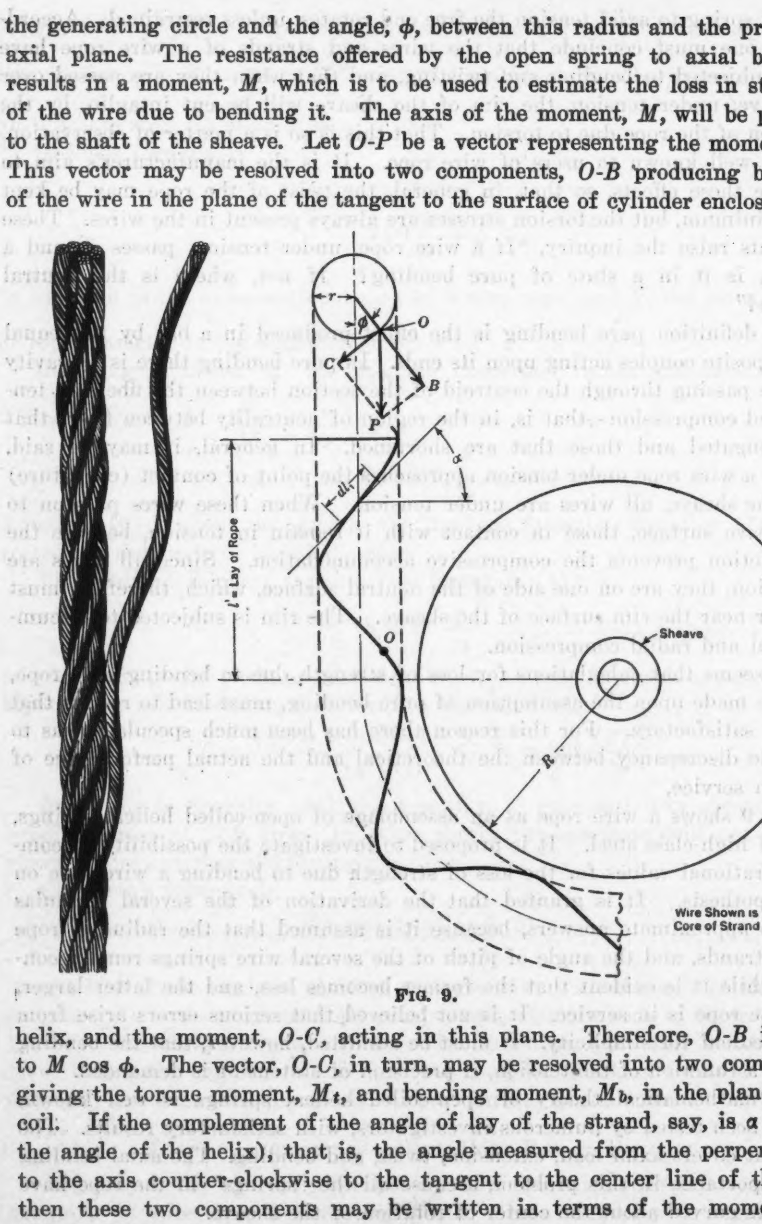

FIG. 9.

helix, and the moment, $O\text{-}C$, acting in this plane. Therefore, $O\text{-}B$ is equal to $M \cos \phi$. The vector, $O\text{-}C$, in turn, may be resolved into two components giving the torque moment, M_t, and bending moment, M_b, in the plane of the coil. If the complement of the angle of lay of the strand, say, is α (equals the angle of the helix), that is, the angle measured from the perpendicular to the axis counter-clockwise to the tangent to the center line of the wire, then these two components may be written in terms of the moment, M: $M_t = -M \sin \phi \cos \alpha$; and, $M_b = M \sin \phi \sin \alpha$.

The bending due to $O\text{-}B$ and M_b must be combined in the usual manner for obtaining maximum and minimum stresses. The bending moment thus considered is measured by $M \sqrt{\cos^2 \phi + \sin^2 \phi \sin^2 \alpha}$.

At this point it is well to recall that the potential energy of bending is given by the expression:

$$U'_b = \frac{M_b^2 l}{2 E I}$$

Likewise, the potential energy of twist is given by:

$$U'_t = \frac{M_t^2 l_1}{2 G_r I_p}$$

in which, I and I_p are the plane and polar moments of inertia; E, the modulus of electricity in tension; G_r, the modulus of rigidity of the steel wire; and l_1, the length of wire in the spring.

The potential energy stored in any short length of the spring, such as $dl_1 = r \frac{d\phi}{\cos\alpha}$, by the bending moment which is $M\sqrt{\cos^2\phi + \sin^2\phi \sin^2\alpha}$, and to the torsion, $M_t = -M \sin\phi \cos\alpha$, may be written:

$$dU' = \frac{r d\phi}{\cos\alpha}\left[\frac{M^2(\cos^2\phi + \sin^2\phi\sin^2\alpha)}{2 E I} + \frac{M^2 \sin^2\alpha \cos^2\phi}{2 G_r I_p}\right] \ldots (13)$$

Integrating between the limits, $\phi = 0$ to $2\pi n$, in which, n is the number of coils, then,

$$U' = \frac{r \pi n}{\cos\alpha}\left[\frac{M^2(1 + \sin^2\alpha)}{2 E I} + \frac{M^2\cos^2\alpha}{2 G_r I_p}\right] \ldots\ldots\ldots\ldots (14)$$

In general, the wires of flexible ropes are circular in cross-section, as compared with profiles of locked and shaped wires for special cables. Using $\frac{\pi (d')^4}{32}$ for I_p and $\frac{\pi (d')^4}{64}$ for I,

$$U' = \frac{8 M^2 l_1}{\pi (d')^4}\left[\frac{2(1 + \sin^2\alpha)}{E} + \frac{\cos^2\alpha}{G_r}\right] \ldots\ldots\ldots\ldots (15)$$

in which, G_r may be defined by $G_r = \frac{E}{2(1+\mu)}$, d' is the diameter of the wire, and μ is Poisson's ratio.

Assume that the rope is fixed at, and beyond, the point of curvature (contact) with the sheave; also, that the work done by the moment in bending the axis of the spring to the radius, ρ, will be equal to the energy stored in the spring; that is, if l, the lay of the rope, is equal to $l_1 \sin\alpha$,

$$\frac{1}{\rho} = \frac{16 M}{\pi (d')^4 \sin\alpha}\left[\frac{2(1 + \sin^2\alpha)}{E} + \frac{\cos^2\alpha}{G_r}\right] \ldots\ldots\ldots\ldots (16)$$

and the moment due to bending a wire shaped as an open helical spring about the center of curvature of the sheave is,

$$M = \frac{\pi (d')^4 \sin\alpha \, E \, G_r}{16 \rho \left[2 G_r (1 + \sin^2\alpha) + E \cos^2\alpha\right]} \ldots\ldots\ldots\ldots (17)$$

which seems to be correct in form, at least, for it meets the well known test that when the elastic curve is an arc of a circle the bending moment is constant. To obtain a value for the stress in the outermost fiber of the wire, in the simplest possible way, for comparison purposes only, $M = \frac{S I}{\frac{d'}{2}}$, or,

$$S = \frac{2 d' \sin \alpha\, E G_r}{[2 G_r (1 + \sin^2 \alpha) + E \cos^2 \alpha]} \quad \ldots\ldots\ldots\ldots\ldots (18)$$

The loss in strength due to bending may be evaluated by ascertaining the amount of axial load on the spring that will give an equal moment. This is done by writing,

$$M = P\, r_1 \sin \alpha \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (19)$$

in which, P is the equivalent axial load, and r_1 is the radius of the strand. This assumption is somewhat justified by the observation of the effect of tension upon wire ropes having fiber centers. The core is compressed, resulting in the formation of grooves, which thus furnishes a partial restriction to the free play of the strand as an open spring having an axis at the center of the rope. Nevertheless, it does not hinder the action of the wires in the strand as open springs about its own center. Furthermore, it is convenient to work with a dimension that is easily attained; for instance, it is a general assumption that the diameter of the strand is one-third the diameter of a 6×19 rope; hence r_1 equals one-sixth of the diameter of the rope. The axial load sustained by a wire of the strand is determined by,

$$P = \frac{\pi (d')^4 E G}{16\, \rho\, r_1 [2 G_r (1 + \sin^2 \alpha) + E \cos^2 \alpha]} \quad \ldots\ldots\ldots (20)$$

The value of P multiplied by the number of wires in the rope gives the loss of strength due to bending.

It will be noticed that Equation (20) takes into account the diameter of the wires, the rope, the radius of curvature, the angle of lay, the modulus of elasticity in tension, and the modulus of rigidity, thereby differing from formulas heretofore proposed. In a broad sense, it may be necessary to determine coefficients by actual tests for different kinds of ropes, just as in the practical application of hydraulic and many other formulas, but as a test of its worth, the loss of strength of the $\frac{3}{4}$-in. rope which was bent around the 6-in. sheave and described in the paper, will next be computed by this formula. It will be recalled that the loss of strength was 3 kips, due to bending.

The actual diameter of the rope as made of cast steel and standard lay, 6×19 construction and with hemp center, was 0.79 in. There were 114 wires of 0.054 in. each. (The lay of the rope is 4.94 in., and of the strand, 1.81 in.). The angle of lay of the strand is 18° 40′, and of the wires in the straight strand, 7° 33′; and, hence, of the wires in the rope, 26° 13′; hence, $\alpha = 63°\ 47'$. The radius of curvature, ρ, is assumed equal to 3.37 in., in the belief that the

mean diameter of the rope, when under tension, is 0.74 in. Furthermore,

$$G_r = \frac{E}{1.3 \times 2} = 11\,000 \text{ kips per sq. in., when } E = 28\,500 \text{ kips per sq. in.}$$

Therefore, by Equation (20), in pounds per wire,

$$P = \frac{3.14 \times 0.054^4 \times 28.5 \times 10^6 \times 11 \times 10^6}{16 \times 3.37 \times 0.125\,[2 \times 11 \times 10^6 (1 + 0.81) + 28.5 \times 10^6 \times 0.19]} = 27.4$$

Multiply by the number of wires, 114 × 27.4 lb. = 3 125 lb., which is in satisfactory agreement with results secured from the test. The writer has experienced considerable satisfaction in estimating the loss of strength due to bending ropes about sheaves and to find the results secured in service were in sufficient agreement so that the riggers and other practical rope users did not smile and turn away when the conversation turned to the strength of ropes, when reeved, say, in the usual wire-rope blocks, an achievement that many sympathetic readers will endorse as being well worth while. It is hoped that the wire rope companies will make the necessary tests to ascertain the general usefulness of this formula, or a comparable one, to the end that the engineer having to do with wire rope may have an accurate estimate of the reduction in strength of the rope due to bending.

The discussion should close at this point because all the foregoing remarks lie in the "battle zone" of wire rope considerations. In order to hold aloft a new target "to shoot at," the following "service factor" formula is offered for discussion. It aims to consider tension and velocity as well as rope-sheave ratios. Since E, the modulus of elasticity of the steel or the rope, is a function of the tension, it is omitted, and the formula is as follows:

$$T_s = k \frac{d}{D} (1 + V)^{0.67} (10 + T)^{0.50} \quad \ldots\ldots\ldots\ldots\ldots (21)$$

in which,

T_s = the total service stress, in kips;
T = the tension on the rope, in kips;
V = velocity, in feet per second; and,
k = a constant for any one type of rope.

Three values may be listed, as follows: For 6 × 7 rope, $k = 300$; for 6 × 19 rope, $k = 250$; and, for 6 × 19 Seale type of rope, $k = 275$.

If V is zero and T is 32 kips, a $\frac{3}{4}$-in. rope on a 6-in. sheave loses 3.6 kips in strength. On a $\frac{3}{4}$-in. sheave, with no velocity, a tension, T, of 12 kips gives a service stress, T_s, of 22 kips; that is, the rope will hold 5 tons when bent on its own diameter without clipping. On the other hand, if a $\frac{3}{4}$-in. rope on a 10-in. sheave has a tension of 5 kips and a velocity of 6 100 ft. per min., the service factor plus the tension will approach the ultimate strength of the rope. In its final form, Equation (21) will probably have a selective coefficient for various conditions of lubrication and exposure. In its present form it is based on the assumption of adequate lubrication and favorable use. If such a service formula is to prevail, the selection of a wire rope for a given duty must be more rational than the guesses of former years.

Equation for an Elastic Curve of a Locked-Coil Cable Supporting a Carrier

The running of sheaves on standing ropes, or the support of moving ropes on guide-sheaves, presents a variation of the bending stress problem, because the radius of curvature of the rope is, in general, greater than the radius of the carriage wheels, or rollers, as the case may be. The following analysis is based on the theory of flexure, because it contemplates locked-coil track cable operating under sufficient tensions to keep the curvature within the limits demanded by classical theory. The analysis results in the determination of such pressures from the helical winding of the several layers of wire, that the internal frictions give properties to the cable, that are similar but of less magnitude than those of a homogeneous bar.

If a portion of such a cable is supposedly taken from a loaded span, it will present the aspect of a beam with fixed ends, in so far as the direction of the axis is concerned, and an analysis to discover the relation of the tension, moment of inertia, modulus of elasticity, load, and other factors may be made as follows: Assume that the cable has sufficient resistance to flexure to form an elastic curve between the points of tangency of the cable to the right and left carriage wheels. Referring to Fig. 8, the summation of the vertical components equals zero and the summation of the horizontal components equals zero; hence,

$$g = T_a \sin \delta_a + T_b \sin \delta_b$$

and,

$$H = T_a \cos \delta_a + T_b \cos \delta_b$$

If $\delta_a = \delta_b$ in Equations (14) and (15),

$$g = 2\,T \sin \delta = 2\,H \tan \delta$$

and,

$$\tan \delta = \frac{g}{2\,H} = \frac{g}{2} \times \frac{1}{H}$$

That is, the section represented by Fig. 8 may be replaced by a free body similar to Fig. 10. It will be noted that either side of the curve assumes

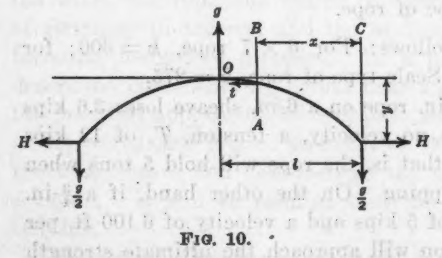

FIG. 10.

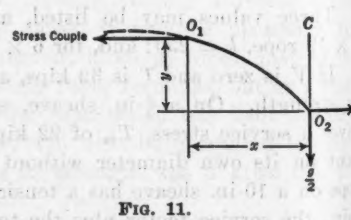

FIG. 11.

a position of equilibrium under the conditions of a vertical load and a horizontal load acting simultaneously. It will be assumed that the conditions controlling the theory of flexure in homogeneous bodies can be applied to locked-coil cable, provided the bending modulus is regarded as variable between the limits of $E = 1\,500$ for no tension and $23\,000$ for a tension of 25 kips.

Since the curvature is slight under ordinary conditions of tension and loading, the approximate formula, $EI \frac{d^2y}{dx^2} = M$, may be used. To apply this equation of the elastic curve consider a free body to the right of Section *A-B* in Fig 10. Taking the summation of moments about O_1 (Fig. 11),

$$M = -\tfrac{1}{2}\, gx + yH = EI\,\frac{d^2y}{dx^2}$$

and,

$$\frac{d^2y}{dx^2} \quad \left(y - \frac{gx}{2H}\right)\frac{H}{EI} \dots\dots\dots\dots\dots\dots (22)$$

For convenience, let $\frac{H}{EI} = i^2$, and $y - \frac{gx}{2H} = u$; then Equation (22) becomes,

$$\frac{d^2y}{dx^2} = i^2\, u = \frac{d^2u}{dx^2}$$

This expression is immediately recognized as a cyclic function because the second differential is a multiple of the function. By integration,

$$\int_0^u \frac{d^2u}{dx^2} = \int_0^u i^2 u = U e^{ix} + W e^{-ix}$$

The equivalent formula, expressed in hyperbolic functions, is,

$$u = U \cosh ix + W \sinh ix$$

and,

$$y = \frac{gx}{2H} + U \cosh ix + W \sinh ix \dots\dots\dots\dots\dots (23)$$

If O_2 is taken as the origin, $x = 0$, $y = 0$, and $U = 0$; let $\frac{g}{2H} = m$; then Equation (23) becomes,

$$y = mx + W \sinh ix \dots\dots\dots\dots\dots\dots\dots\dots (24)$$

and,

$$\frac{dy}{dx} = m + i\, W \cosh ix \dots\dots\dots\dots\dots\dots\dots (25)$$

If $x = l$, Equation (25) becomes zero; therefore,

$$W = -\frac{m}{i \cosh il} \dots\dots\dots\dots\dots\dots\dots\dots (26)$$

Substituting in Equation (24),

$$y = mx - \frac{m \sinh ix}{i \cosh il} \dots\dots\dots\dots\dots\dots\dots (27)$$

or,

$$y = \frac{g}{2H}\left[\, l - \frac{\tanh \sqrt{\frac{H}{EI}}\, l}{\sqrt{\frac{H}{EI}}} \,\right] \dots\dots\dots\dots (28)$$

The general equation may be written,

$$y = \frac{g}{2\,H}\left[x - \frac{e^{-ix}}{i\,e^{i}l}\right] = \frac{g}{2\,H}\left[x - \frac{1}{i\,e^{ilx+e}}\right] \ldots\ldots\ldots (29)$$

in which, $i = \sqrt{\frac{t}{EI}}$ to simplify typography.

The slope of the tangent at the point, xy (Fig. 8), is,

$$\frac{dy}{dx} = \frac{g}{2\,H}\left[1 + \frac{e^{ix}}{eil}\right] \ldots\ldots\ldots\ldots\ldots (30)$$

If $x = l$,

$$y = \frac{g}{2\,H}\left[l - \frac{1}{ie^{2il}}\right] \ldots\ldots\ldots\ldots\ldots (31)$$

and,

$$\frac{dy}{dx} = \frac{g}{2\,H}\left[1 + \frac{1}{e^{2il}}\right] \ldots\ldots\ldots\ldots\ldots (32)$$

If the curve (Fig. 8) is assumed to be a circular arc, the radius of curvature can be found for the arc, $O_1\,O_2$, (Fig. 11), for any value of $\varDelta$ and any slope of cable, provided $\tan \varDelta = \frac{g}{H}$.[8]

Conclusion

There are many phases of wire rope—its attributes and its manufacture—that may well remind one of the philosopher's "category" and the "unknowable." Modern wire rope represents the best in mining and selecting iron ore and limestone, in the making of coke, in blast-furnace operations, in the selection of scrap, in the control of the "heat" of the open hearth, in the human equation of the melter, in the boss roller, in the soaking shift foreman, in the cleaning house boss, the wire drawer, the tester, the manila rope manufacturer, the annealer, the spooler, the men on the stranding and laying machines, the warehouse, salesmen, and numerous chemists, physicists, executives, and in the use of the power of wealth; all co-ordinated to produce the best product that can be made within the price the purchaser will pay. Every one of these items and more have uncertain factors that may not have been safeguarded against or eliminated, but will be when the art of steel and wire-rope making ceases to be extremely competitive and is directed toward the creation of quality products.

In any symposium for the improvement of the service life of wire the issue must be joined by considering this question: Can the processes involved in the manufacture, the rolling of rods, the drawing of wire, the laying of wire rope, and its uses, be brought under critical technical control to the end that a behavior of wire rope in service may be predicted with assurance?

[8] For tables of stress in locked-coil cables, see *Transactions*, Am. Soc. C. E., Vol. 92 (1928), pp. 940-948.

Of course, some ropes fail to render service. The wonder is that so many prove satisfactory in spite of abuse and neglect. It is probable that the improvement of wire rope will be the result of a gradual advance in the control of the basic operations of its manufacture and use.

APPENDIX I

Notation

The following will serve as a general guide for the definition of all symbols used in the paper:

b = unit bending stress.
d = diameter of a wire rope; d' = diameter of a single wire.
e = base of natural logarithms.
f = coefficient of friction.
g = total weight applied by a tramway carriage.
i = substitution factor $= \sqrt{\frac{H}{EI}}$ (Equation (23)).
k = a constant varying with the type of rope (Equation (21)).
l = length along a rope; l' = length of lay, or the pitch of a single wire; l_1 = length of a twisted wire in the straight distance, l' (Fig. 2).
m = a substitution factor $= \frac{g}{2t}$ (Equation (24)).
n = number of coils.
r = radius of a rope; r' = radius of a circular bar, or a single wire. (Distance between the two arms of a wire rope loop.) r_1 = radius of a strand.
s = a sheave.
u = a substitution factor $= y - \frac{gx}{2H}$.
w = suspended weight; w' = unit dead load weight of rope. Distance from the origin of ordinates to a point, $x\,y$ (see Fig. 8).
x = horizontal distance from origin to point, $x\,y$.
y = deflection of a wire rope.
A = cross-sectional area of a rope; A' = section area of a single wire.
B = reduced distance between arms of a wire rope around a sheave, supporting a load (Fig. 7).
C = circumference of a helix circle.
D = diameter of a sheave.
E = modulus of elasticity; E', in tension; E_b in bending.
F = length of circumference of helix circle (see Fig. 2).
G = a force normal to a cable at any point; G_r = modulus of rigidity.
H = horizontal components of tension; H_a and H_b = components of T_a and T_b, respectively.
I = planar moment of inertia; I_p = polar moment of inertia.
K = ratio of rope diameter to wire diameter, $= \frac{d}{d'}$.
L = length of wheel-base of a tramway carriage.
M = bending moment.
P = an axial load on a wire.

$R =$ radius of curvature of bends in rope, or semi-diameter of a sheave; $R_w =$ radius of carriage wheels; $R_t =$ radius of tread of a carriage wheel; R' (see Table 5); $R_c =$ radius of curvature of a bent rope.

$S =$ strength of wire rope; $S' =$ strength of a single wire; S'_s = strength of a strand.

$T =$ tension in a wire rope; $T_c =$ tension at the crest of a sheave; $T_s =$ total service strength; $T' =$ load on a single straight wire (Fig. 1); $T_t =$ load on a single twisted wire (Fig. 2); $T_a =$ rope tension toward the left; $T_b =$ rope tension toward the right.

$U =$ a substitution factor in Equation (23).

$U' =$ potential energy; U'_b, of bending; and U'_t, of twisting.

$V =$ velocity of a wire rope.

$W =$ a substitution factor in Equation (23).

$\alpha =$ angle between a curved center wire and a wire twisted about it (Fig. 3), or the angle of pitch of an open spring.

$\beta =$ angle between the axis of a rope and a curved center wire (Fig. 3).

$\gamma =$ angle of contact between rope and sheave.

$\delta =$ angle between forces, T and H.

$\theta =$ angle between principal axial plane and radius to Point 0 (Fig. 9), on open helical spring.

$\Delta =$ angle of intersection of the tangents of a bent wire rope $= \delta_a + \delta_b$ (Fig. 8).

$\phi =$ angle of lay of the outside wires of a strand of rope.

$\mu =$ Poisson's ratio.

$\rho =$ radius of curvature.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

PAPERS

LAKE CHAMPLAIN BRIDGE[1]

BY CHARLES M. SPOFFORD,[2] M. AM. SOC. C. E.

SYNOPSIS

A chronological narrative of the planning, design, and construction of the bridge that crosses Lake Champlain from Crown Point, N. Y., to Chimney Point, Vt., is given in this paper. After an historical summary outlining the preliminary legislative steps necessary, the subjects of traffic counts, estimates, finances, and proposed toll rates are discussed.

A comparison of various types of bridges studied for this crossing and the reasons for adopting the continuous truss type are presented, followed by a description of the finished bridge itself, surveying methods of field location, specifications, loadings, materials, concrete, etc. Construction problems encountered on the substructure and superstructure are treated under separate headings.

HISTORICAL

Since early times Lake Champlain has been an important traffic route between the valleys of the St. Lawrence and the Hudson Rivers and long before the construction of railways or highways the lake carried considerable freight traffic. During the struggle between France and England over control of the North American Continent the possession of the lake became of utmost importance both to the French and the English and strong forts were erected by both nations at Crown Point and at Ticonderoga. In Revolutionary times, equal importance attached to the control of the lake, and the capture by Ethan Allen and the "Green Mountain Boys" of Fort Ticonderoga by a night crossing of the lake forms a notable event in American history. While the lake has been a traffic route for centuries, it has also been a serious barrier to free intercourse between the State of Vermont, which was formerly

NOTE.—Written discussion on this paper will be closed in March, 1932, *Proceedings*.

[1] Presented at the meeting of the Structural Division, Boston, Mass., October 10, 1929.

[2] Hayward, Prof. of Civ. Eng., Mass. Inst. Tech., Cambridge; Cons. Engr. (Fay, Spofford & Thorndike), Boston, Mass.

a part of New York State, and New York State itself—a barrier, 120 miles in length, without a bridge. Communication across it has been by infrequent ferry lines operated in the season of open waters only, approximately 8 months per year. In many cases the ferries were not operated during the night and, in some cases, the approaches were hazardous. Occasionally, they were prevented from operating by bad weather. During the period between the formation of thick ice and its breaking up, occasional routes across the ice are available for vehicles.

The bridge described in this paper is the first and only vehicular bridge crossing the lake in a distance of 120 miles. Its westerly approach is located a few miles above the Village of Crown Point, N. Y., at the Crown Point Reservation. The latter is a State Reservation containing the ruins of Fort St. Frederick and Fort Amherst, and the Champlain Memorial Lighthouse. It is being developed as a public park with bathing facilities, recreation piers, and park sites.

The bridge forms a connecting link between the main north and south highway routes on either side of the lake and also establishes a new traffic route between the Adirondacks, the White and the Green Mountains, and the Maine, New Hampshire, and Northern Massachusetts cities and coast resorts.

Popular demand for a bridge across the lake to replace the ferry-boat service, existed for a long time and finally "came to a head" in the Vermont General Assembly by the passing in 1923 of an Act creating a commission to inquire into the feasibility and possibility of joint action with New York State to bridge the lake. In February, 1925, the New York Assembly also established a commission to investigate such a bridge from the standpoint of New York and thereafter the two State Commissions acted together in making a preliminary investigation. The report[3] of this Joint Commission, submitted in 1926, discusses, in a general way, several possible bridge locations. It gives statistics as to traffic, points out the inconvenience of ferries, and recommends that the New York Commission be continued to complete its investigations and that additional funds be provided for borings and other studies. As a result, the New York Commission was continued and given a grant of \$25 000. The Vermont Commission was automatically continued.

The 1927 report[4] of the Joint Commission contained data giving the results of borings at five possible locations, a report of the geologic conditions at these locations, and preliminary designs with estimates of cost of bridges at the proposed sites. The engineering features of this report were prepared by Roy G. Finch and J. A. L. Waddell, Members, Am. Soc. C. E. The geologic report was made by C. A. Hartnagel, Assistant Geologist of the State of New York.

[3] Legislative Document (1926) No. 93, of the State of New York, entitled, "Preliminary Report of the Joint Legislative Commission on Bridge Connections Between the States of New York and Vermont Across Lake Champlain."

[4] Legislative Document (1927) No. 59, of the State of New York, entitled, "Final Report of the Joint Legislative Commission upon Bridge Connections Between the States of New York and Vermont."

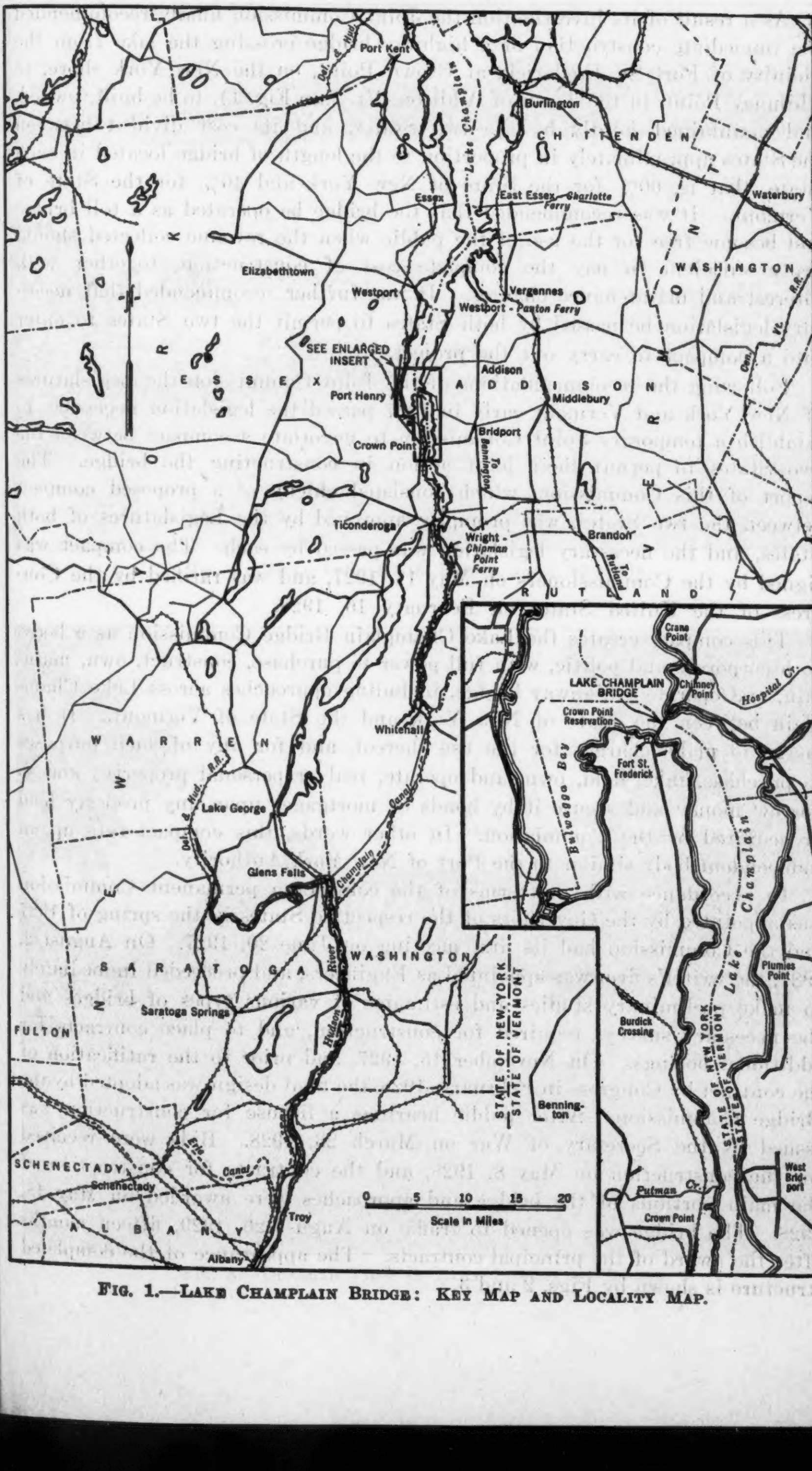

FIG. 1.—LAKE CHAMPLAIN BRIDGE: KEY MAP AND LOCALITY MAP.

As a result of its investigation the Joint Commission finally recommended the immediate construction of a highway bridge crossing the lake from the vicinity of Fort St. Frederick, at Crown Point, on the New York shore, to Chimney Point, in the Town of Addison, Vt. (see Fig. 1), to be built, owned, and maintained jointly by the two States, and its cost divided between the States approximately in proportion to the length of bridge located in each State, that is, 60% for the State of New York and 40% for the State of Vermont. It was recommended that the bridge be operated as a toll bridge and become free for the use of the public when the revenue collected should prove sufficient to pay the complete cost of construction, together with interest and maintenance charges. It was further recommended that necessary legislation be passed by both States to permit the two States to enter into a compact to carry out the project.

Following the recommendations of the Joint Commission the Legislatures of New York and Vermont early in 1927 passed the legislation necessary to establish a temporary Joint Commission to negotiate a compact between the two States to permit their joint action in constructing the bridge. The report of this Commission, which consisted chiefly of a proposed compact between the two States, was promptly approved by the Legislatures of both States, and the necessary legislation was passed by each. The compact was signed by the Commissioners on May 11, 1927, and was ratified by the Congress of the United States on February 16, 1928.

This compact creates the Lake Champlain Bridge Commission as a body, both corporate and politic, with full power to purchase, construct, own, maintain, and operate a highway bridge, including approaches across Lake Champlain between the State of New York and the State of Vermont. It has power to make charges for the use thereof, and for any of such purposes to purchase, take, hold, own, and operate, real or personal property; and to borrow money and secure it by bonds or mortgages upon any property held or acquired by the Commission. In other words, this compact sets up an independent body similar to the Port of New York Authority.

In accordance with the terms of the compact a permanent Commission was appointed by the Governors of the respective States in the spring of 1927 and the Commission had its first meeting on June 20, 1927. On August 2, 1927, the writer's firm was appointed as Engineers, and proceeded immediately to make preliminary studies and estimates of various types of bridges and the necessary surveys required for construction, and to place contracts for additional borings. On November 15, 1927, and prior to the ratification of the contract by Congress in February, 1928, the final design was adopted by the Bridge Commission. After public hearings a license for construction was issued by the Secretary of War on March 23, 1928. Bids were received for the construction on May 8, 1928, and the contracts for construction of the main portions of the bridge and approaches were awarded on May 18, 1928. The bridge was opened to traffic on August 26, 1929, fifteen months after the award of the principal contracts. The appearance of the completed structure is shown by Figs. 2 and 3.

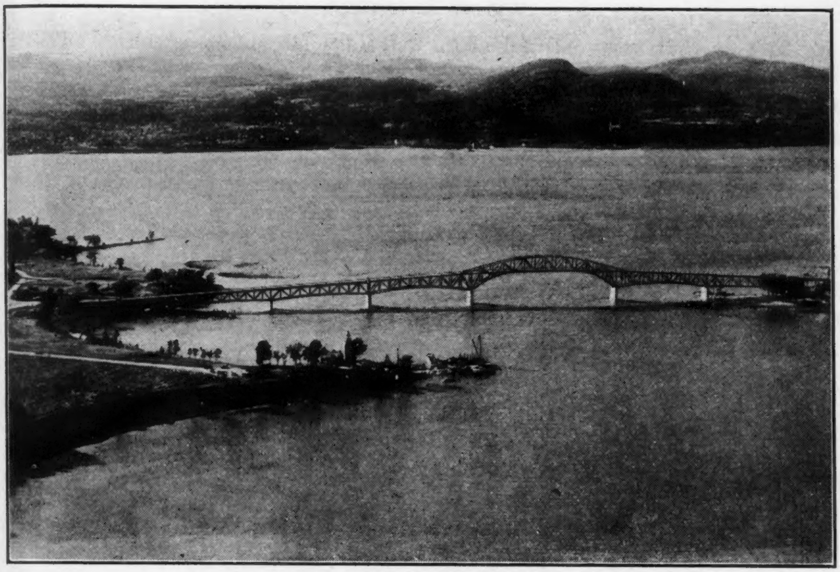

FIG. 2.—LAKE CHAMPLAIN BRIDGE FROM THE AIR.

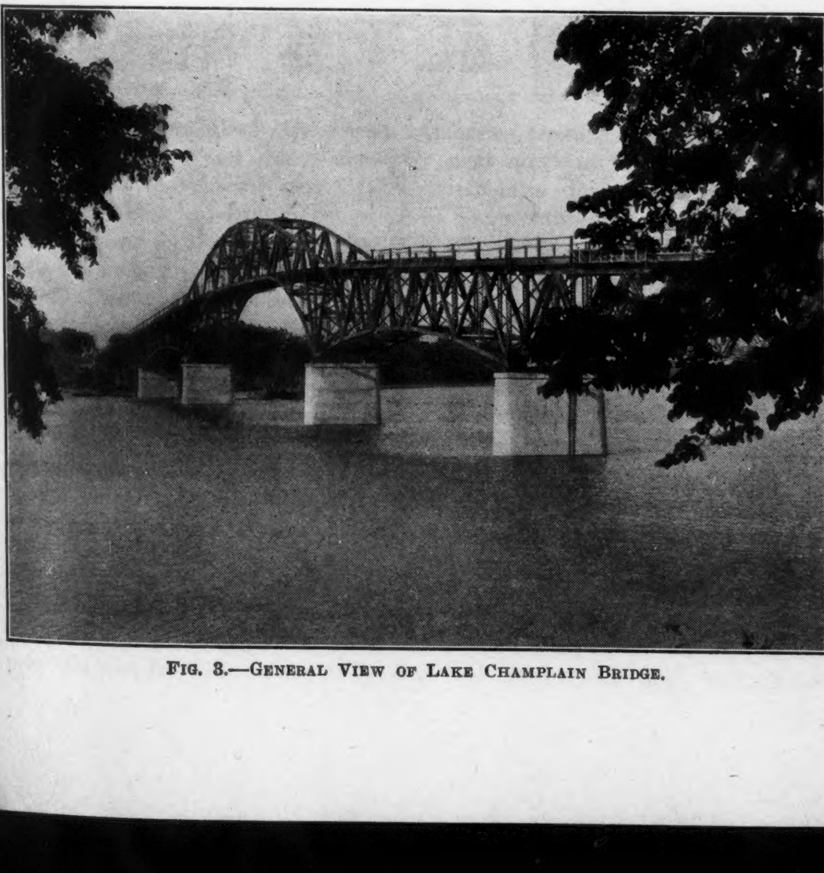

FIG. 3.—GENERAL VIEW OF LAKE CHAMPLAIN BRIDGE.

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Estimated Traffic

The estimates of annual vehicular traffic for the years 1930 to 1951 are given in Table 1. These data were based on the following considerations: Statistics of ferry traffic given by the Joint Legislative Commission in its reports on the Lake Champlain Bridge project show that for the five ferries from the bridge site southerly to the Wright's-Chipman Point Ferry the total traffic carried during the season, May to November, 1925, was "more than 75 000 automobiles." These same five ferries, together with the two ferries (Westport-Panton and Essex-Charlotte) next north of the bridge site, are known to have "carried over 80 000 automobiles during the six months' operating season" of 1926. The normally severe winters in this region cause ice conditions on the lake such that it is not possible for ferries to maintain open channels and to operate during the winter months. These conditions and the light patronage both contribute to limit the navigation season to about six months of the year, beginning approximately May 1, and ending about November 1, although under favorable seasonal conditions some of the ferries may continue operation into, and even through, the month of November.

TABLE 1.—Estimated Annual Vehicular Traffic for the Years 1930–1951

Year of operation	Year ending June	Number of vehicles	Annual percentage, increase	Triennial percentage, increase
1 (part)	1930	30 500	...	....
2	1931	60 000		....
3	1932	69 000	15.0	....
4	1933	77 500	12.3	
5	1934	85 500	10.3	42.5
6	1935	93 000	8.8	....
7	1936	100 000	7.5	
8	1937	106 500	6.5	24.5
9	1938	112 500	5.6	....
10	1939	118 000	4.9	....
11	1940	123 000	4.2	15.5
12	1941	127 500	3.7	....
13	1942	131 500	3.1	....
14	1943	135 000	2.7	10.
15	1944	138 000	2.2	....
16	1945	140 500	1.8	....
17	1946	142 500	1.4	5.5
18	1947	144 000	1.1	....
19	1948	145 000	0.7	....
20	1949	146 000	0.7	2.5
21	1950	147 000	0.7	....
22	1951	148 000	0.7	....

Nearly all the ferries operate only during daylight hours. Some, however, will give night service on signal at advanced rates. The ferries having the longer runs, as, for example, those operating out of Burlington, Vt., follow a regular schedule and this is true also of some of the short-run ferries. Other short-run ferries are operated only on signal. Ferry operation is subject to interruption and delay due to fogs, head winds, and rough water, and to occasional break-down of the motive power of the boats.

In view of the foregoing considerations had a bridge been available at the proposed site in 1926, it was estimated: (*a*) That of the more than 80 000

cars carried by these seven ferries during that operating season (all of which ferries are within about twenty miles of the bridge site), at least 60% or fully 50 000 cars would have used the bridge; (*b*) that other traffic would have been diverted from more distant points (the Burlington ferries and from around the ends of the lake) amounting to, at least, 10 000 cars; and (*c*) that 10 000 additional cars would have used the bridge during the winter and at other times when the ferries were not operating. Hence, a total of 70 000 cars was assumed as the probable traffic which would have crossed a bridge at the Crown Point-Chimney Point site in 1926; but to be conservative, 60 000 cars were taken in making up financial estimates as the number which would cross the bridge in its first normal operating year ending June 30, 1931.

No reliable data were available as to the growth from year to year of the ferry traffic across Lake Champlain. It was necessary, therefore, to base estimates of the probable growth of traffic upon statistics of traffic in general and in the districts bordering the bridge, and also to allow for effect of changed conditions resulting from the construction of the bridge.

TABLE 2.—GROWTH OF MOTOR VEHICLE TRAFFIC ON STATE HIGHWAYS

Item No.	Location	Years	Length of period, in years	Percentage increase
	MOTOR VEHICLE REGISTRATION			
1........	United States....................................	1923–27	4	53
2........	New York State....................................	1923–27	4	61
3........	Vermont....................................	1923–27	4	51
	TRAFFIC CENSUS			
4........	Massachusetts (two periods and at numerous points)..	1918–21 1921–24	3 3	66
5........	New York State (section toward the west of Syracuse)..	1923–27	4	52
6........	Saranac Lake to Lake Placid....................	1923–27	4	79
7........	Warrensburg to Chestertown....................	1923–26	3	69
8........	Saratoga Springs to Lake George....................	1923–26	3	60

Table 2 shows the growth in automobile registration, and in traffic on the Massachusetts highways as well as on the New York State highways westerly of Syracuse, which may be considered as representing normal traffic growth. Values are also included in the tabulation for certain of the highways in the Adirondack Region which are more representative of the growth in tourist travel (Items 6, 7, and 8).

Statistics of traffic on the New York highways in the Adirondack Region must be viewed in the light of the fact that, during the years 1925 to 1930, extensive road construction brought about sub-normal traffic conditions in many instances and super-normal conditions in others. By the time the bridge was to be opened to travel, on September 1, 1929, it was expected, however, that the State highway construction program would be nearly

finished; and that the automobile highway from New York City to Plattsburg, N. Y., and Montreal, Que., Canada, along the westerly shore of Lake Champlain, the most direct route from New York City to these northern points, would be entirely completed. It was assumed that these improvements would be reflected in an increased traffic in the districts served by the bridge and also on the bridge itself.

Another factor which was expected to have a decided influence upon the bridge traffic was the fact that the Crown Point Reservation—within which is located the New York approach to the bridge, and which contains the ruins of the historic forts, Amherst and St. Frederick—is being developed as an historic shrine and public park by the State Conservation Department. The development program provides for the building of a recreational pier and boat landing on the shore of the Reservation of the Champlain Memorial Lighthouse, the building of bathhouses and the improvement of the bathing beach, the extension of the museum, the construction of adequate buildings for the shelter of visitors, and provisions of ample parking facilities for motorists.

It was also confidently expected that the opening of the bridge would itself stimulate travel in this region such that the growth of traffic across the bridge would proceed, at least for the first few years, at a substantially higher rate than that on long established highways where for many years conditions have remained substantially unchanged. Tourist travel in general is increasing faster than normal travel, and as this bridge would form a connecting link between regions of tourist travel, the Adirondacks on the one hand and the Green Mountains, the White Mountains, and Maine resorts on the other hand, the effects of this rapidly increasing tourist travel would unquestionably be reflected in the bridge traffic. Moreover, the opening of a new and improved route across important waterways has been found in numerous cases to be accompanied by a marked increase in traffic, as shown by the examples that follow.

Traffic as a whole over the Delaware River was increased 70% in one year's time by the opening of the Philadelphia-Camden Bridge. With the opening of the Carquinez Strait Bridge across an arm of San Francisco Bay the bridge traffic of 1927 was 71% greater than the traffic on that important California highway route carried by ferry in 1926, whereas the normal growth in traffic over the 7-year period prior to the opening of the bridge showed an average increase of only 15.6 per cent.

At the Peace Bridge over the Niagara River at Buffalo, N. Y., where the ferry traffic had been 350 000 cars per year, the bridge traffic during the first year's operation was 1 407 272 vehicles, or four times the traffic on the ferry which the bridge replaced.

In the case of the Holland Tunnel beneath the Hudson River between New York and New Jersey, during its first year of operation ending November 12, 1928, it is estimated by the Chief Engineer that approximately 45% of the year's business was new traffic which would not have crossed the river except for the improved traffic facilities provided by the tunnel, and that only the remaining 55% was diverted from the ferries.

In view of the probable stimulation in traffic due to the foregoing causes, it was assumed that for a few years after its opening the travel over the bridge would increase rapidly and at a rate greater than the average increase in traffic on the highways generally of Vermont and New York. After a time this increase would be less rapid and in eight or ten years the increase might be expected to correspond more closely with the growth of traffic on the highway systems of these two States.

Financing and Insurance

The estimated and actual receipts and expenses from the time of opening the bridge to June 31, 1931, are shown in Table 3.

TABLE 3.— Estimated and Actual Toll Receipts

Month	Actual Earnings by Month		Earnings as Estimated by Engineer*	
	Monthly	Total from August 27, 1929 (cumulative)	Monthly	Total from September 1, 1929 (cumulative)
1929:				
August (5 days)	$3 062.75		......	
September	16 393.35		$6 000	
October	7 726.25		4 000	
November	4 116 50		2 000	
December	2 110.00	$33 408.85	2 000	$14 000
1930:				
January	$821.25		$1 000	
February	921.25		1 000	
March	1 934.75		1 000	
April	3 292 50		2 000	
May	4 981.25		4 000	
June	7 562.50	$52 922.35	7 500	$30 500
July	12 967.50		11 000	
August	18 234.75		11 000	
September	10 616.00		9 000	
October	6 582.00		6 000	
November	3 987.75		3 000	
December	2 438.00	$107 748.35	2 000	$72 500
1931:				
January	$1 286.00		$1 000	
February	881.00		1 000	
March	1 704.00		1 000	
April	3 390.00		2 000	
May	5 247.25		5 000	
June	6 964.75	$127 221.35	8 000	$90 500

* Engineer's report of May 24, 1928, used as a basis of financing.

Serial bonds, bearing interest at $4\frac{1}{4}\%$ and redeemable at any interest date upon thirty days' notice, at varying premiums, were issued to finance this project. They are legal and tax exempt in New York and Vermont for public officials and bodies of each State and of its municipal sub-divisions, for insurance companies, savings banks, and other fiduciary bodies. In the opinion of counsel they are also exempt from the Federal income tax.

Estimates of gross revenues and net earnings, together with payments of interest at the rate of 4% and retirement on the bond issue of $1 000 000, are shown in Table 4.

The revenues are based on an average toll of $1 per car, including all passengers. This rate is less than the average toll rate charged by the several Lake Champlain ferries. The estimates indicate that by 1952 the accumulated surplus will be sufficient to retire the bonds then outstanding ($420 000) and to pay principal and interest on the $200 000 advanced by the States of New York and Vermont.

TABLE 4.—ESTIMATED FINANCIAL SHOWING FROM BRIDGE OPERATIONS

Year ending June 30	Gross revenues from tolls	Operating expense	Net earnings	Bond interest and retirement	Bonds outstanding after retirement paid July 1 of year in question
1929	....	....	....	$40 000	$1 000 000
1930	$30 500	$14 000	$16 500	40 000	1 000 000
1931	60 000	15 000	45 000	40 000	1 000 000
1932	69 000	15 500	53 500	40 000	1 000 000
1933	77 500	16 000	61 500	40 000	1 000 000
1934	85 500	16 500	69 000	40 000	1 000 000
1935	93 000	17 000	76 000	40 000	1 000 000
1936	100 000	17 500	82 500	40 000	1 000 000
1937	106 500	18 000	88 500	40 000	1 000 000
1938	112 500	18 500	94 000	40 000	1 000 000
1939	118 000	19 000	99 000	40 000	1 000 000
1940	123 000	19 500	103 500	60 000	980 000
1941	127 500	20 000	107 500	89 200	930 000
1942	131 500	20 500	111 000	87 200	880 000
1943	135 000	21 000	114 000	85 200	830 000
1944	138 000	21 500	116 500	83 200	780 000
1945	140 500	22 000	118 500	81 200	730 000
1946	142 500	22 500	120 000	79 200	680 000
1947	144 000	23 000	121 000	77 200	630 000
1948	145 000	23 500	121 500	75 200	580 000
1949	146 000	24 000	122 000	73 200	530 000
1950	147 000	24 500	122 500	71 200	480 000
1951	148 000	25 000	123 000	79 200	420 000
1952	149 000	25 500	123 500	....	....
Total	$2 669 500	$459 500	$2 210 000	$1 381 200	....

The bridge (except for its filled approaches) is insured for use and occupancy and for damages by fire, lightning, flood, tornado, earthquake, collapse, explosion, riot, malicious action, and other insurable hazards.

TYPE OF BRIDGE AND REASONS FOR ITS ADOPTION

While no limit was set upon the capital cost of the proposed bridge, in order to sell the bonds it was necessary that the bridge should show a probable revenue from tolls sufficient to cover carrying charges and also to amortize the investment in a reasonable time, which was fixed at thirty years by the Commission. The width of the roadway was fixed by the compact, and the clear width and rise of the channel span were determined by navigation requirements as established by the United States Army Engineers.

The Commission and its engineers were in agreement that the bridge should have as pleasing an appearance as possible consistent with the foregoing. The historic importance of the site and the fact that the bridge would be conspicuous for many miles on account of its height made its appearance of special importance. Borings and test piles disclosed that the soil overlying the bed-rock was extremely soft and that the bridge piers would

have to be carried to bed-rock which over a part of the site occurs at a depth of 100 ft. below low-water level. After consultation with Army officials, it was decided to provide a vertical clearance at the channel span of 90 ft. above standard low water (Elevation 92.5) for a width of 186 ft., and a clearance of 73 ft. above the same level for a width of 300 ft. Fig. 4 shows

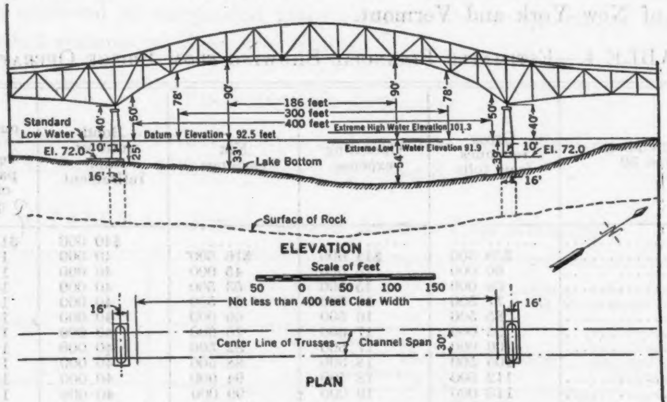

FIG. 4.—CHANNEL SPAN, LAKE CHAMPLAIN BRIDGE.

the clearance at various points in the channel span. The total width of the lake at this water level is approximately 1 500 ft.

In selecting the type of bridge consideration was given to the relative advantages and disadvantages of the following types:

(*a*) End-supported truss bridge;
(*b*) Cantilever truss bridge;
(*c*) Continuous truss bridge; and
(*d*) Suspension bridge.

An arch bridge either of reinforced concrete or steel was not considered economically possible in view of the expense of carrying unbalanced arch thrusts to the bed-rock at its great depth below the arch springing lines.

In comparing the availability of the various types of truss bridges the method of erection had to be given careful consideration. Navigation requirements made it essential that the channel span should be erected without the use of falsework. In some truss bridges this has been done by erecting a complete span on a lighter and hoisting it in place, but such a method was not seriously considered for this bridge in view of the lack of suitably located anchorage basins in the vicinity of the site. In consequence, all studies of truss bridges were based upon erecting the center span by cantilevering each half of it from the adjoining span. This method would not involve special difficulty, or greatly increased expense in the case of the cantilever or continuous type of structure; but for an end-supported center span, some of the truss members would have to be increased in size.

Moreover, in the engineer's judgment, the end-supported type, if designed with due regard to economy (with a shorter center span than that actually

used for the Lake Champlain Bridge), would prove much less pleasing in appearance than either the continuous or the cantilever type and, also, would require somewhat more expensive center piers, owing to the greater pier width necessary to provide landing space for abutting trusses than that required for the support of a continuous truss. If end-supported spans were to be given the same outline as the continuous spans adopted, the trusses would be far from economical in outline. A further complication affecting the appearance of a bridge with end-supported trusses was the gradient which would require deck trusses for part of the crossing and through trusses over the channel. The writer found it impossible to sketch any simple span design that was at all satisfactory in appearance.

A cantilever bridge of the usual statically determined type did not seem especially appropriate, particularly in view of the necessarily short suspended span which would be proportionably shorter than is generally considered advisable for cantilever bridges. Some study was given to this type, but it was not possible to discover any important advantages as compared with a continuous structure while it would have the disadvantage of additional expansion joints in the floor and of greater and more rapid deflection particularly in the center span. Noticeable deflection of any bridge, especially when the deflection occurs rapidly, is objectionable both from the standpoint of the users of the bridge (a particularly important factor in a highway bridge), and of maintenance. It was, therefore, the engineer's desire to produce as rigid a bridge as possible with due regard to first cost and maintenance charges. A cantilever bridge that is not statically determined, such as the Queensboro Bridge in New York City, is, in reality, continuous. The comparisons of this paper do not refer to such a bridge, but only to the usual type of statically determined cantilever bridge.

The continuous truss type has all the advantages of the cantilever type except that of statical determination, and is also economical of material, especially when the dead stresses are large compared with the live stresses as in the case of a highway bridge with a concrete floor and with spans as long as those of this bridge. The lack of statical determination requires additional mathematical investigations on the part of the designers, but involves no special theoretical difficulties, merely increasing the labor of making the necessary computations. The deflection of a continuous span is less in amount and occurs with less rapidity than that of a cantilever span, this being an important element in favor of the continuous bridge. Moreover, the continuous type can be given a more pleasing appearance, consistent with economy, than any of the other types of truss bridges.

One objection sometimes raised to continuous bridges is that settlement of foundations causes serious changes in truss stresses, but with piers supported on bed-rock, as in the Lake Champlain Bridge, this objection does not exist. Moreover, in the case of yielding foundations most of the settlement occurs during the construction of the piers themselves, and does not affect the truss stresses. Unequal settlement of the piers after the erection of the trusses and the establishment (by jacking) of the desired end reactions would affect only that part of the truss stresses due to the dead loads carried by

the trusses at the time the ends are jacked. These loads would include the weight of the steel trusses and bracing and such portions of the floor system as might be found convenient for erection purposes; that is, unless the unequal settlement is so great as to leave the trusses suspended over one of the piers or abutments, a contingency which should be guarded against by proper design.

With both end-supported and cantilever truss spans rejected in favor of continuous truss spans, it became necessary to compare this type of structure with the suspension span type.

Detailed estimates were made of a continuous span structure and two suspension bridges. One suspension bridge had a center span of 700 ft., with two flanking spans of 350 ft. each and two 185-ft. approach spans, and the other had a center span of 1 000 ft., with two flanking spans each with a length of 350 ft. The estimated cost of the shorter of these suspension spans proved to be somewhat greater than that of the continuous truss type. Moreover, its appearance was not entirely satisfactory since the main structure, consisting of the center span and the two flanking spans, would not have sufficient length to extend from shore to shore at all stages of the lake level thus giving it the effect of not being quite long enough to accomplish its purpose. The Commission and the engineers agreed that such a structure would not look as well as the continuous span finally adopted. The other suspension bridge was long enough to extend from shore to shore without the use of approach spans, and would have made an attractive bridge, except for the fact that such a span appeared of unwarranted length. The cost of such a structure, however, would be so much greater than that of a bridge with continuous trusses that its adoption could not be considered.

A design involving the use of continuous span trusses for the greater part of the structure, therefore, was adopted finally with the approval of the Commission and the engineers. The writer and his associates believe that the structure as designed has a pleasing appearance, is economical, and will prove satisfactory in operation and maintenance. The outline of the trusses and the general dimension of the entire structure are shown in Fig. 5.

General Description

The bridge is a high-level structure approximately 2 190 ft. in length, of which 1 500 ft. more or less is over the water. The ascending gradients of $5\frac{1}{2}\%$ upward from each end to the channel span are connected by a parabolic vertical curve. The main part of the bridge consists of one end-supported riveted-truss structure; two continuous riveted-truss structures (one with two spans and the other with three spans); and eight plate-girder viaduct spans (three on the New York shore and five on the Vermont shore), each having a length of approximately 50 ft. The girders of the viaduct spans were riveted together after the dead load was in place, making them continuous for the live loads, and certain of the columns supporting the viaduct spans are hinged at each end. The viaduct spans are reached by filled approaches, one approximately 400 ft. in length and the other approxi-

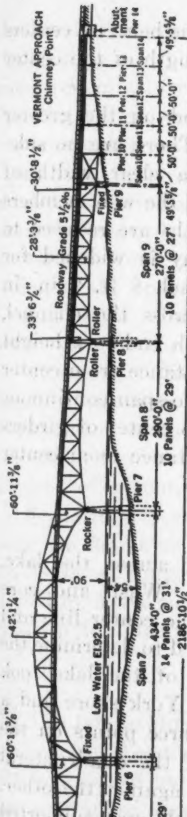

FIG. 5.—ELEVATION OF LAKE CHAMPLAIN BRIDGE SHOWING DEPTHS OF TRUSSES

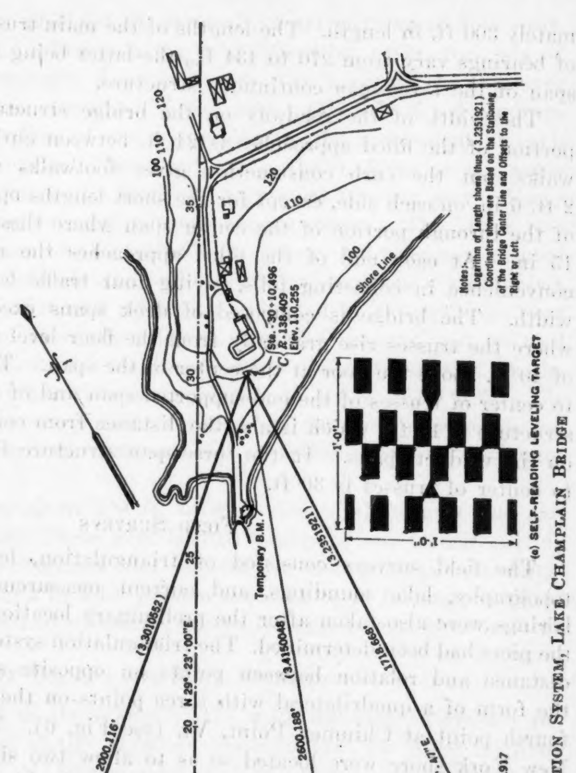

FIG. 6.—TRIANGULATION SYSTEM, LAKE CHAMPLAIN BRIDGE

mately 300 ft. in length. The lengths of the main truss spans between centers of bearings vary from 270 to 434 ft., the latter being the length of the center span of the three-span continuous structure.

The width of the roadway on the bridge structure and on the greater portion of the filled approaches is 24 ft. between curbs. There are no sidewalks, but the curb construction gives footwalks with a clear width of 2 ft. 6 in. on each side, except for the short lengths opposite the web members of the through portion of the center span where these widths are reduced to 15 in. At each end of the filled approaches the roadway is widened for convenience in collecting tolls, giving four traffic lanes each 8 ft. 9 in. in width. The bridge is composed of deck spans except across the channel, where the trusses rise gradually from the floor level at each end to a height of 40 ft. above the floor at the center of the span. The distance from center to center of trusses of the end-supported span and of the two-span continuous structure is 18 ft., which is also the distance from center to center of girders on the viaduct spans. In the three-span structure the distance from center to center of trusses is 30 ft.

Field Surveys

The field surveys consisted of triangulation, leveling across the lake, topography, lake soundings, and current measurements. Wash and core borings were also taken after the preliminary location of the center line and the piers had been determined. The triangulation system used to determine the distance and relation between points on opposite shores of the lake took the form of a quadrilateral with three points on the New York shore and a fourth point at Chimney Point, Vt. (see Fig. 6). The three points on the New York shore were located so as to allow two sides of the quadrilateral to be taped precisely and used as base lines, checking one against the other. The taping of the base lines was done with a 100-ft. steel tape, supported near its center and having a 12-lb. pull applied by a spring balance at one end. Corrections were computed for the slope, temperature, and sag. Each base line was more than 1 000 ft. in length, and was measured twice with differences of 0.001 ft. and 0.012 ft. in the lengths obtained. Each angle was turned six times forward and six times backward with a transit, reading to 10 sec., first with the telescope direct and then inverted. These results were checked later by a different observer. The measured angles were adjusted to balance the various triangles, the usual correction being 1 sec. or 2 sec.

Reciprocal leveling with a precise level was used to transfer the elevations across the lake a distance of about 1 500 ft. The precise level used had three leveling screws and a bubble with an angular value of 7 sec. per division. Bench-marks were set close to each shore, at about the same elevation. The level was set near one of the bench-marks, and a series of readings were taken across the lake. The position of the level and target was then reversed, and another series of observations were taken in the opposite direction.

A self-reading home-made target, shown by Fig. 6 (*a*), was used for this work. It had four rows of alternate black and white spaces each 0.1 ft. high,

the two outer rows being offset 0.1 ft. vertically from the two middle rows, thus aiding the observer in interpolating to hundredths. The two sets of measurements taken in opposite directions were then averaged to eliminate the effect of refraction and curvature of the earth. One set of rod readings was also taken on the surface of the water close to the instrument at each shore as a check on the results. The results from the readings on the water checked the average of the results from the reciprocal leveling more closely than the water level could be read without a hook-gauge. The latter was considered for use in place of the reciprocal leveling, but was not adopted owing to the uncertain effect of wind and the need of building a stilling-box at each shore. The curvature and refraction indicated by the readings were about 0.05 ft. during the morning readings, but this amount was reduced by afternoon. No check on the results of either the triangulation or the leveling has been made other than an approximate check made before the construction of the center spans and tests made by running lines across the ice which gave a check in both distances and elevations within a few hundredths of a foot.

Soundings were taken about 300 ft. to each side of the center line and near the shores and docks. These were located mostly by one transit cutting across range lines. The directions and velocities of the currents were determined by floats made from 2 by 2-in. wooden studs, 13 to 16 ft. long, weighted so as to float vertically and to submerge, except for approximately about 1 ft.

Live Loading and Unit Stresses

The live loading used in the design of the main members of trusses and girders was Loading *H*-15 of the Standard Specifications for Steel Highway Bridges issued by the U. S. Department of Agriculture,[5] and as revised by mimeographed supplement. This loading gave a uniform load of 525 lb. per ft. per girder or truss, combined with a concentrated load of 24 500 lb. applied at a point where it would produce maximum stress in the member under consideration. The live loads used in the design of the floor system and columns or hangers supporting the floor-beams consisted of one 20-ton truck moving along the bridge in any part of the roadway, or two 20-ton trucks side by side moving in the same or opposite directions. The dimensions and load distributions of these trucks are those given in the specifications previously mentioned.

Impact and wind stresses were also computed in general in accordance with the foregoing specifications. The unit stresses used in the design of the main truss members were 16 000 lb. per sq. in. for axial tension and for bending on the extreme fiber, and $15\,000 - \dfrac{50\,L}{r}$ for axial compression, with corresponding values for other unit stresses. In designing the members of the floor system allowable unit fiber stresses of 18 000 lb. were used instead of the 16 000 lb. used for the main truss members.

[5] *Bulletin 1259*, U. S. Dept. of Agriculture.

Materials

Steel.—Rolled steel, rivet steel, cast steel, and forgings were required to conform to the standard specifications of the American Society for Testing Materials for carbon steel, as follows:

Structural and Rivet Steel: Open-hearth (Serial Designation A7-24);
Cast Steel: Open-hearth, electric, or crucible (Serial Designation A27-24);
Steel Forgings: Open-hearth, or electric (Serial Designation A20-27);
Reinforcing Trusses for Concrete Floor: Open-hearth billet structural steel (Serial Designation A15-14); and
Other Reinforcement Bars: Either the same as for reinforcing trusses or rail steel (Serial Designation A16-14).

Paint.—Exposed metal work was given one shop coat of red lead and two field coats of tinted white lead. The coloring pigment used in the first field coat consisted of a small quantity of lampblack giving this coat a brown color. Considerable study was given to the second field coat to ensure a color that would harmonize with the landscape and prevent the bridge from being too conspicuous. The color finally selected was made by mixing from 13 to 13½ lb. of raw umber in oil with 100 lb. of white lead, and adding a small quantity of ferrite yellow until the exact color was obtained.

The red lead, white lead, linseed oil, and drier used in the paints were required to conform with the specifications of the Federal Specifications Board, as follows:

Red Lead: U. S. Standard Specifications No. 11, as corrected and revised by mimeograph circular of December 15, 1897, for grade containing not less than 95% Pb_3O_4.
White Lead: U. S. Standard Specifications No. 5.
Linseed Oil: U. S. Standard Specifications No. 4.
Driers: U. S. Standard Specifications for Liquid Paint Drier No. 20.
Turpentine: American Society for Testing Materials (Serial Designation D13-24).

Concrete.—Various grades of concrete were required for different parts of the work, and approximate parts by volume for these various grades were stated in the specifications as well as the maximum gallons of water, inclusive of the water contained in the aggregate per bag of cement. Slump tests for the various grades were also specified, depending upon the proportions of the mix, these tests to be made in accordance with the Tentative Method of Tests for Consistency of Portland Cement Concrete of the American Society for Testing Materials (Serial Designation D138-25T). These various requirements are stated in Table 5.

Coarse and fine aggregate used in the concrete came from iron mines at Mineville, N. Y., and were obtained at a depth of from 1 500 to 2 000 ft. below the surface. This material consisted of crushed stone from which the iron had been separated by a magnetic separator, less than 10% of the iron remaining in the rock after separation. The aggregate was transported to Port Henry by rail and thence to the bridge site in scows. Test cylinders of concrete were made regularly throughout the progress of the work and

tested in the Cambridge Laboratories of the Massachusetts Institute of Technology. These tests showed the concrete to be unusually strong.

In view of the fact that much of the pier concrete had to be deposited in water at a considerable depth, the question arose as to what effect the hydrostatic pressure would have both on the time of setting and the strength of the concrete. An investigation of this question in 1928 and 1929 revealed that concrete similar to that used on this bridge, deposited in water at a pressure of 50 lb. per sq. in. (corresponding to a head of 115 ft.) develops a somewhat higher compressive strength in periods between 3 and 28 days than similar concrete setting under atmospheric conditions.

TABLE 5.—SLUMP TEST. REQUIREMENTS FOR VARIOUS CLASSES OF CONCRETE.

Designation (Class)	Maximum water, per sack of cement	Approximate Parts by Volume			Maximum slump, in inches
		Cement	Fine aggregate	Coarse aggregate	
A	7.0	1	2	4	4
A (in walls)	...	...	...	...	6
B	7.7	1	2.5	5	4
C	7.0	1	2	4	3
D	6.5	1	1.8	3.6	3
*E**	...	1	1.5	2	3

* The proportion of water for this mixture to be determined in the field.

Substructure

The character and depth of the material overlying the bed-rock are shown by the record of borings, given in Fig. 7. These tests were made in 1927. In addition, the data from borings made in 1926 were available to contractors, for examination. Five test piles were also driven. The overlying material was found to be so soft that a pile foundation would not be feasible; hence it was necessary to sink the piers to bed-rock, the depth of which reached a maximum of approximately 100 ft. below the low-water level of the lake.

Geological reports, core borings, and outcropping ledges showed the bed-rock to be of Chazy limestone formation of homogeneous structure and well fitted to carry any loads which could be put upon it by the bridge. The investigations indicated that the surface of the rock would be reasonably level and free from disintegration, and these conclusions were verified by the conditions found during construction.

All abutment piers and viaduct footings are of concrete. The piers and certain of the viaduct footings bear on bed-rock. The abutments and other viaduct footings are supported on good material without piling. The general type of all the piers (except Pier 9, which is on shore), is shown by the channel pier illustrated in Fig. 8. Pier 9 was laid "in the dry."

Piers 3 to 8, inclusive (see Fig. 5), had to be built in water, and it was specified that all of them, except the two channel piers (Piers 6 and 7), should be built by the open coffer-dam method. Bids were obtained for building the two latter piers by either the pneumatic caisson or open coffer-dam process.

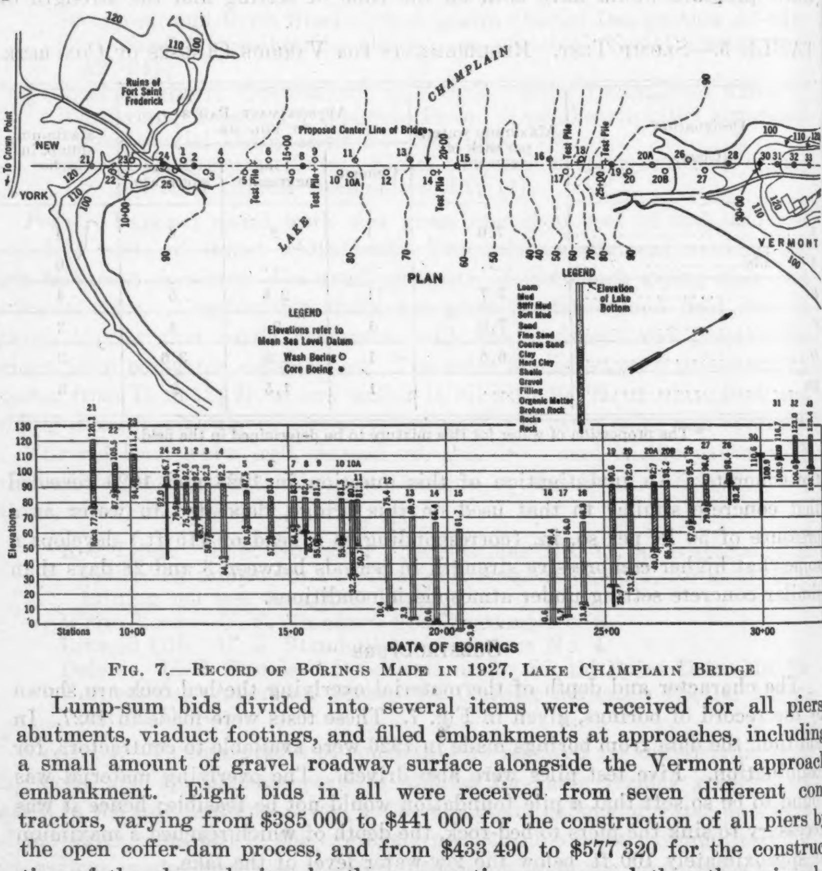

FIG. 7.—RECORD OF BORINGS MADE IN 1927, LAKE CHAMPLAIN BRIDGE

Lump-sum bids divided into several items were received for all piers, abutments, viaduct footings, and filled embankments at approaches, including a small amount of gravel roadway surface alongside the Vermont approach embankment. Eight bids in all were received from seven different contractors, varying from \$385 000 to \$441 000 for the construction of all piers by the open coffer-dam process, and from \$433 490 to \$577 320 for the construction of the channel piers by the pneumatic process and the other piers by the open coffer-dam process.

The itemized bids for Piers 6 and 7 showed a variation from \$176 444 to \$295 000 by the open coffer-dam process and from \$249 340 to \$349 000 by the pneumatic process. The accepted bidder's figure for these two piers was \$220 653, approximately \$27 000 lower than the lowest bid for the same piers by the pneumatic process. The second lowest bidder submitted the lowest bid for these two piers, but put in a much higher bid for the other piers, thereby making his total bid \$4 485 higher than that of the accepted bidder. It should be noted that the second lowest bid for Piers 6 and 7 by the coffer-dam method was \$72 896 lower than the lowest bid for these piers by the pneumatic process.

Consideration of the bids for Piers 6 and 7 show clearly the financial advantages of the open coffer-dam process for the particular conditions which existed at these piers, namely, soft material overlying bed-rock at an approxi-

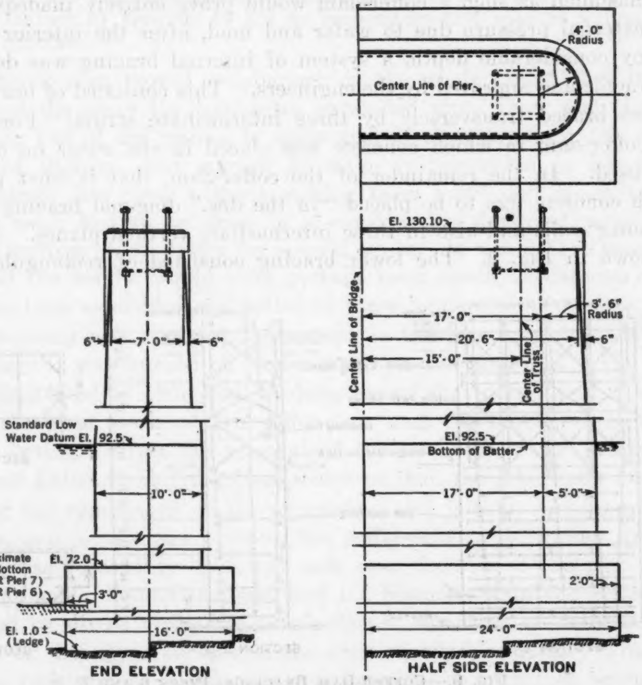

FIG. 8.—DIMENSIONS OF CHANNEL PIERS.

mate distance below extreme low water of 100 ft., and below extreme high water of 109 ft., in a fresh-water lake without rapid changes in water level and with comparatively little current. So far as the writer is aware, these bridge piers are the deepest yet constructed by the open coffer-dam process with a single-wall coffer-dam and with a deep concrete seal deposited under water.

Wooden piles varying in length from 60 to 80 ft. were driven around the site of each pier and capped with 12 by 12-in. timbers placed approximately 8 ft. above the low-water level. On this capping a rough, working floor was placed. Waling pieces, 12 by 12 in., were fastened on this floor and similar ones were fastened approximately at the water level. These waling pieces served as guides for the steel sheet-piling forming the coffer-dam and were located horizontally so as to provide a clearance of approximately 4 ft. all around the concrete required by the plans for the portion of the pier below the lake bed. The sheet-piling formed a continuous wall weighing approximately 32 lb. per running ft. It had a width of 16 in., with corrugations of 4 in. The maximum length of pile used was 98 ft., consisting of two pieces spliced

by bolting in lengths of 40 ft. and 58 ft. Piling was driven to bed-rock without difficulty by a steam hammer weighing 7 000 lb. In no case were obstacles encountered, such as boulders or driftwood.

Inasmuch as such a coffer-dam would prove entirely inadequate to resist the external pressure due to water and mud, after the interior was dredged to any considerable depth, a system of internal bracing was devised by the contractor and approved by the engineers. This consisted of heavy horizontal frames braced transversely by three intermediate struts. For the part of the coffer-dam in which concrete was placed in the water no other bracing was used. In the remainder of the coffer-dam, that is, that part of it in which concrete was to be placed "'in the dry," diagonal bracing was used in the outer walls and also in three intermediate vertical planes. This bracing is shown in Fig. 9. The lower bracing consisted of rectangular frames of

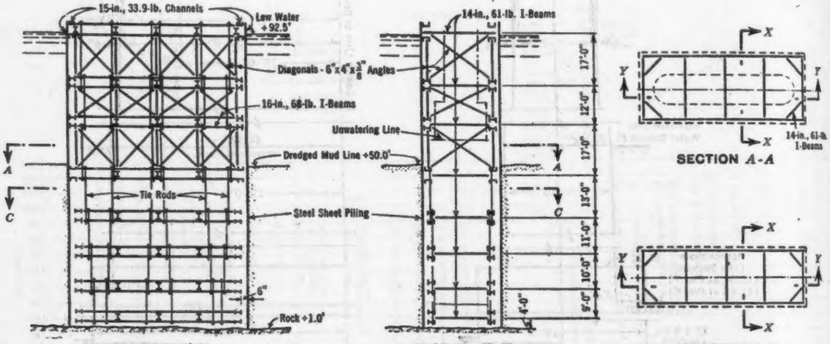

FIG. 9.—COFFER-DAM BRACINGS, PIERS 6 AND 7.

double 16-in., 68-lb. I-beams riveted together at the corners by knee-braces with three cross-struts in each frame, each consisting of a single 14-in., 61-lb. I-beam. The horizontal frames for the upper part of each coffer-dam was similar, but made of somewhat lighter members.

Horizontal frames had to be left in place in the part of the pier in which concrete was laid in water. The 4-ft. clearance between coffer-dam walls and the specified dimension lines of the pier was sufficient to permit these frames to be placed entirely outside the required cross-section of the pier. The three cross-struts in each frame, however, passed through the piers.

For the upper part of the pier the horizontal frames were removed during construction, but the cross-struts and diagonals were embedded in the concrete. The diagonals, however, were cut off at the upper ends of each panel.

The contractor's plan for placing the bracing below the mud line (which was accepted with slight modification for practically all piers), was as follows:

(a) After excavating approximately 12 ft. of mud from the coffer-dam, place all the horizontal bracing in this space in the form of a nest, and wedge the upper horizontal frame in place against the sides.

(b) Proceed with the excavation and, when conditions permit, lower the remaining frames until the upper one is at its desired level, and then wedge it in place. Proceed in this manner until all the bracing is in place, connecting the various sets of horizontal frames by tie-rods to act as suspenders and spacers during the process.

The foregoing plan proved satisfactory on the whole except that in the case of one of the piers the contractor excavated more than was planned before wedging one of the frames in place and, as a consequence, the sheet-piling buckled slightly—just enough to make it impossible to lower the remaining frames into place without cutting them apart and re-assembling them. This work had to be done in a considerable depth of water and required the services of six divers for several weeks. It would have been less expensive and the results would have perhaps been equally satisfactory had all the frames been assembled and bolted in place by divers.

The sheet-piling was easily pulled, except in the pier in which buckling occurred, where it was burned off under water at the mud-line. The mud was readily excavated by clam-shell buckets, except that portions of it which adhered to the steel work of the coffer-dam and the bracing had to be loosened by jetting. After the removal of the mud, the water within the coffer-dam was found to be freer from sediment than the lake water outside.

Following the completion of the excavating and jetting, soundings were made within, and jet borings without, the coffer-dam to determine whether the bed-rock was reasonably level, not only over the site of the pier itself, but for a reasonable distance outside, and the bottom within the coffer-dam was examined by divers to determine whether it was clear of sediment and whether any of the upper surface of the rock required removal. The condition of the rock surface on all piers was found to be such that no further preparation of these surfaces was necessary. As a matter of fact the condition of these surfaces was especially satisfactory since they had been scored somewhat by glacial action and, in some cases, had pot-holes, thus ensuring good bond between concrete and bed-rock. In no cases were the surfaces tilted appreciably.

The concrete was placed in position by a patented, bottom-drop bucket with a capacity of 1 cu. yd. and made of $\frac{1}{2}$-in. steel. At the bottom of the bucket double flap doors were attached to the bail by a chain. When suspended by the bail these doors were closed. When the bucket reached bed-rock, or the upper surface of the concrete previously laid, the bail would slide down, releasing the flap doors. This also released a clutch which engaged a lug on the side of the bail, preventing the doors from closing when the bucket was raised to permit the concrete to flow out. It was found that the cubic yard contained in the bucket spread on a level surface in a circle of 10 to 12 ft. in diameter.

Experience with depositing concrete in this manner shows that for the most satisfactory results the bucket should be level full; otherwise when the bucket submerges, the rush of water will wash the cement from the upper portion of the concrete in the bucket. Moreover, the concrete should not

be too dry. For the concrete in these piers a slump of 3 to 4 in., as determined by the standard slump test, was required. The concrete was laid from the center of each space between cross-braces toward the sides, in order to allow the laitance to flow toward the outer sides of the coffer-dam beyond the pier dimension lines.

After the under-water concrete that extended to grade, Elevation 72 (about 20 ft. below the low-water line), was placed, the coffer-dam was unwatered and the remainder of the concrete laid "in the dry," the sheet-piling providing a satisfactory dam for this stage of the work.

SUPERSTRUCTURE

General Description and Unit Prices.—The superstructure consists of riveted truss and plate-girder viaduct spans all of carbon steel, supporting a concrete floor. The make-up of the various main members of the superstructure involves no unusual features and will not be described. The maximum stress in any truss member occurs in the bottom chord of the 270-ft. end-supported span. In the continuous spans the maximum stress in any truss member is tension and occurs over any intermediate pier.

Itemized bids for the steel, concrete floor, curbs, and broken stone on approaches were received. The bids on important items varied as follows:

Structural steel trusses, etc., erected and painted, per pound....	\$0.06925	to	\$0.084
Floor system, viaduct, etc., per pound...	\$0.066	to	\$0.0788
Bridge flooring complete	\$98 000	to	\$141 840

The successful bidder for the superstructure presented the lowest aggregate bid on the basis of estimated quantities, at \$0.06925 per lb. for all structural steel and \$98 000 for the bridge flooring.

The amount of structural steel as estimated by the engineers in asking for bids was 6 000 000 lb. The actual weight, as shown by final estimates for payment, was 5 883 727 lb., or approximately 60 tons less than the original estimate.

Stress Computations.—In designing the continuous trusses preliminary stress computations were made, using the values given in Griot's tables of influence data for shear and moment for continuous beams of constant moment of inertia.[6]

The principal dimensions of the structural members in Spans 6, 7, and 8, are shown in Fig. 10. For these spans, the values of the dead reactions, as obtained at Piers 5 and 8 from the preliminary design, were used as final values, a clause being introduced into the specifications stipulating that these predetermined reactions should be secured by the use of hydraulic jacks fitted with gauges and with the ratio between inches raised and reactions applied determined by actually raising the ends of the trusses before the laying of the concrete floor. Prior to the final establishment of the reactions by jacking, the Method of Least Work was used to determine the

[6] "Kontinuierliche Trager Tabellen Griot", Second Edition, pub. by von Aschmann & Scheller, Zurich, Switzerland.

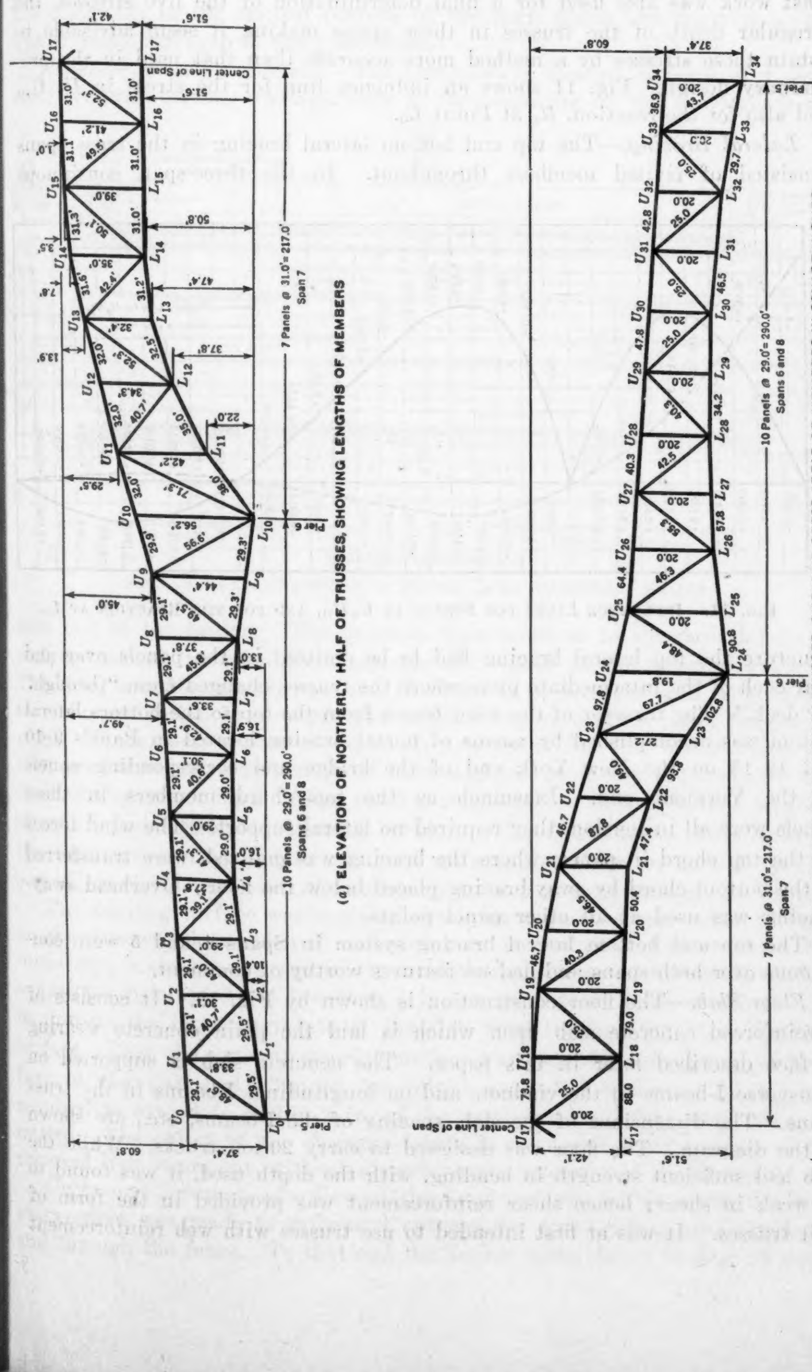

FIG. 10. PRINCIPAL DIMENSIONS, SPANS 6, 7, AND 8.

dead-load reactions due to the steel work and the temporary tracks and concrete forms which were on the span prior to the jacking. The method of least work was also used for a final determination of the live stresses, the irregular depth of the trusses in these spans making it seem advisable to obtain these stresses by a method more accurate than that used in the preliminary design. Fig. 11 shows an influence line for the stress in U_9 U_{11}, and also for the reaction, R_L at Point L_0.

Lateral Bracing.—The top and bottom lateral bracing in the truss spans consisted of rivited members throughout. In the three-span continuous

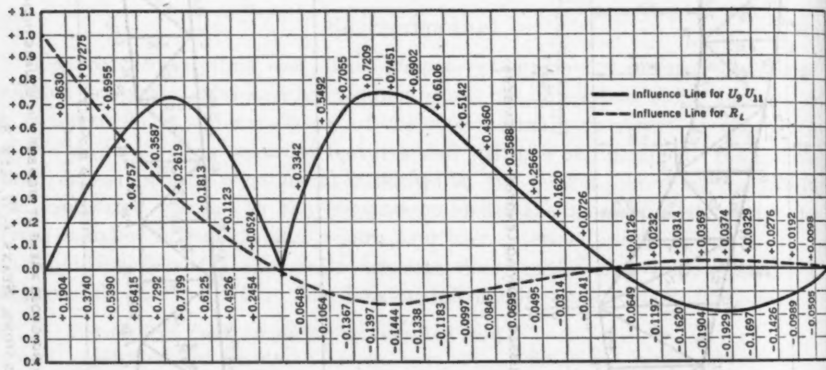

FIG. 11.—INFLUENCE LINES FOR STRESS IN U_9 U_{11}, AND FOR THE REACTION AT L_0.

structure the top lateral bracing had to be omitted in the panels over and near each of the intermediate piers where the trusses changed from "through" to "deck." The transfer of the wind forces from the top to the bottom lateral system was accomplished by means of portal bracing located in Panels 9–10 and 12–13 on the New York end of the bridge and corresponding panels on the Vermont end. Inasmuch as the top chord members in these panels were all in tension, they required no lateral support. The wind forces on the top chord in panels where the bracing was omitted, were transferred to the bottom chord by sway-bracing placed below the floor. Overhead sway-bracing was used at all other panel points.

The top and bottom lateral bracing system in Spans 4 and 5 were continuous over both spans and had no features worthy of comment.

Floor-Slab.—The floor construction is shown by Fig. 12. It consists of a reinforced concrete slab upon which is laid the plain concrete wearing surface described later in this paper. The concrete slab is supported on transverse I-beams in the viaducts and on longitudinal I-beams in the truss spans. The dimensions of the slab, spacing of the I-beams, etc., are shown in the diagram. The floor was designed to carry 20-ton trucks. While the slab had sufficient strength in bending, with the depth used, it was found to be weak in shear; hence shear reinforcement was provided in the form of unit trusses. It was at first intended to use trusses with web reinforcement

along the entire length of each unit, such as have been used in other bridges, but it was found that a material saving in cost could be made by omitting the diagonals in the central portion of the span where the concrete alone was of ample strength to carry the shear.

Concrete Wearing Surface.—The Commission decided in favor of a concrete roadway surface corresponding to the surfacing on the State roads at

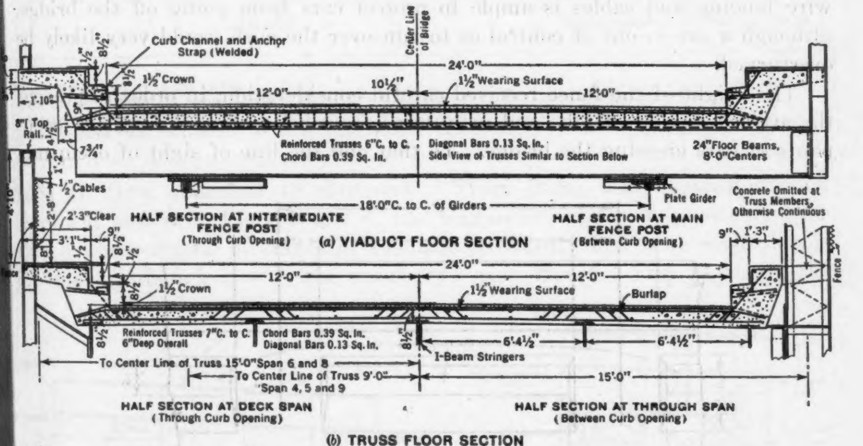

FIG. 12.—CROSS-SECTIONS OF FLOOR, LAKE CHAMPLAIN BRIDGE

each end of the bridge. The question then arose as to the desirability of making use of the reinforced concrete slab itself as the wearing surface. This idea was abandoned, however, in order to protect the slab from wear, involving reduction in strength and possible exposure of the steel reinforcement. These factors seemed especially important for this bridge with its steep gradients and long periods of snow and ice, requiring the frequent use of chains on motor cars. After careful consideration it was decided to construct a special patented wearing surface. The Commission itself secured the right to use this surface and paid all royalties therefor, as well as the cost of special supervision of the laying by a representative of the owner.

The wearing surface was made by obtaining a flush mortar upper surface on the concrete floor-slab and embedding in it (before it had acquired its initial set) a special cleavage fabric consisting of burlap with strands having a uniform spacing of not less than $\frac{1}{4}$ in., or more than $\frac{3}{8}$ in., small enough to prevent the large aggregate of the slab from bending with the wearing surface. Then the concrete wearing surface was poured before the upper surface of the slab had obtained its initial set. The cleavage fabric was required to extend to within 2 in. of the curb, and lap-joints were not permitted. The object of the cleavage fabric is to permit the top layer to be removed if it wears out, without disturbing the integrity of the floor-slab.

Curbs and Fencing.—In view of the height of the bridge above the lake every effort was made to safeguard automobiles from riding over the curb and through the fence. To that end the double curbs shown in Fig. 12 were

provided. From abutment to abutment the fencing was made of steel wire mesh attached to wire cables at top and bottom, the cables being stretched into position between fence-posts. In addition, a structural steel channel top rail was used, this being connected to the fence posts, except in the through portion of Span 7 where it was connected to the truss web members. The writer believes that the protection afforded by the double curb and the wire fencing and cables is ample to protect cars from going off the bridge, although a car so out of control as to run over the curb would very likely be overturned.

The height of the fence received careful consideration, in order to prevent the attractive view of lake and mountains from being cut off from the occupants of cars crossing the bridge. To that end the line of sight of occupants

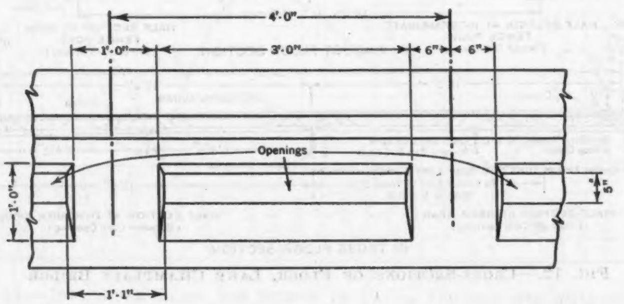

FIG. 13.—ELEVATION SHOWING TYPICAL CURB ELEVATIONS.

of motor cars of average height was studied, and the decision was finally reached to locate the top of the rail at a height of 4 ft. 6 in. above the concrete curb, with the top of the fencing 14 in. below the top of the rail, thus affording a clear space between bottom of rail and top of upper cable of approximately $11\frac{1}{2}$ in. Views can also be obtained through the meshes of the fence. The fencing is of the chain-link type and consists of a fabric of galvanized aluminum wire slightly less than No. 6 gauge, with 2-in. mesh and with both edges knuckled. The shop coat of paint was applied by dipping the fencing while still warm from galvanizing.

Drainage and Snow Removal.—In view of the fact that the curb which also forms a footwalk on either side of the bridge would add considerably to the weight of the bridge if solid, the curb openings shown in Fig. 13 were provided. These serve not only to reduce the weight of curb and amount of concrete, but they also give ample provision for drainage and improve materially the appearance of the bridge as seen in side view. It is also expected that they will help keep the bridge floor free from snow which, if not too damp, will to some extent be blown off the bridge through these openings. The openings were made uniform in size throughout the length of the viaduct and the truss spans to simplify the form work. Unit quantities in the floor are as given in Table 6.

Expansion Joints.—Because of the continuity of the truss and viaduct spans, it was possible to reduce the number of expansion joints (which are always troublesome in highway bridges) to four in all in a total length of

TABLE 6.—QUANTITIES OF MATERIAL IN THE FLOOR, PER LINEAR FOOT OF BRIDGE

Material	Truss spans	Viaduct spans
Wearing surface, in cubic yards	0.11	0.11
Other concrete floor, in cubic yards	0.87	0.97
Reinforcing trusses, pounds	153	68
Reinforcement rods, in pounds	97	141

2187 ft. from abutment to abutment. These joints were located at each abutment and over Piers 5 and 8, the maximum expansion occurring over Pier 8 where a total clearance of 15 in. is provided. The same expansion is provided (although it was not necessary) over Pier 5. Fig. 14 shows the

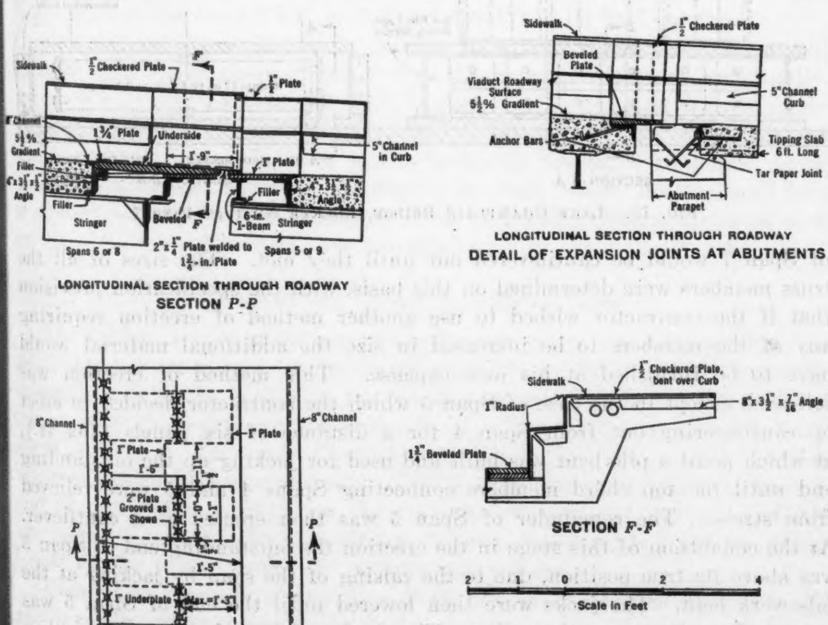

FIG. 14.—EXPANSION JOINTS, LAKE CHAMPLAIN BRIDGE.

details of the expansion joints at these two points. Attention may be called to the fact that the expansion plates consist of flat plates roughened by transverse strips welded thereto instead of the checkered plates commonly used.

Rocker and Roller Bearings.—In order to reduce the friction in the truss bearings and to prevent rusting, the rollers and rockers are enclosed in steel boxes filled with Albany grease and with sides that can be removed for replacing the grease if this becomes necessary. This method of protecting the rollers was used on the Queen City Bridge across the Merrimac River at Manchester, N. H., and an examination several years after its construction showed that the Albany grease had not hardened and that no rusting of the rollers had occurred. Fig. 15 gives details of one of the rocker nests on Pier 7.

Erection.—In designing the superstructure it was assumed that Spans 4, 5, 6, 8, and 9 would be erected on falsework and that the projecting ends

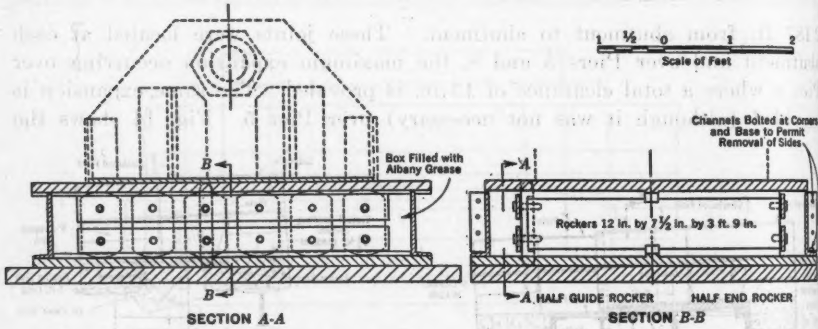

FIG. 15.—LAKE CHAMPLAIN BRIDGE, ROCKER NEST AT PIER 7.

of Span 7 would be cantilevered out until they met. The sizes of all the truss members were determined on this basis, with the specification provision that if the contractor wished to use another method of erection requiring any of the members to be increased in size the additional material would have to be furnished at his own expense. This method of erection was followed, except in the case of Span 5 which the contractor decided to erect by cantilevering out from Span 4 for a distance of six panels (162 ft.), at which point a pile bent was built and used for jacking up the outstanding end until the top chord members connecting Spans 4 and 5 were relieved from stress. The remainder of Span 5 was then erected as a cantilever. At the conclusion of this stage in the erection the outstanding end of Span 5 was above its true position, due to the raising of the span by jacking at the falsework bent. The jacks were then lowered until the end of Span 5 was supported on its appropriate pier. The final position of this end was then established by jacking at the pier. This method of erection involved an increase in the size of a few of the main members of Span 4, but the cost of this additional material was comparatively small.

The method of erection adopted for Span 7 required careful computations of the horizontal movements of Panel Points U_{17} and L_{18} at the cantilevered ends of Span 7 due to cambering and to the weight of steel and erection track. The deflection due to the traveler when located at one of the pro-

jecting ends was also determined. Computations also had to be made of the position of the bottom panel point, L_{24}, under varying conditions of load and temperature to determine the proper location of the rocker nests at this point with the rockers vertical. These computations were made by the contractor, and the decision was reached to set the rockers at this pier vertical and 2 in. forward (south) of their position under dead load and normal temperature.

The horizontal deflections of Panel Points L_0 and L_{34} were also computed to determine the proper position of the base plates at these points, which were specified to be located so that the roller nests should be central at mean temperature. The horizontal movements of U_0 and U_{34} were also computed to determine the amount of expansion to be provided in the expansion joints at these points.

The rollers, wedges, and shims provided at L_0 and L_{34} of Spans 6, 7, and 8 were temporarily omitted, and the truss shoes were supported directly on the base plates, planed surfaces being required for the sole plates on the shoes and the base plates to permit sliding during erection. In consequence, these panel points were located 20 in. below their final height. Panel Point L_{24} was also located 3.25 in. forward (south) of its final position.

For closure, Panel Points L_0 and L_{34} were both raised by jacking, and Span 8 with the projecting end of Span 7 was moved forward horizontally. As these processes were carried out it became possible to insert the previously omitted truss members in an unstressed condition. After these members were in place, L_0 and L_{34} were raised farther by means of calibrated jacks, until the calculated end reactions due to the steel trusses, bracing, and floor members, together with the erection track and such concrete forms as were in place to time of jacking, were obtained.

Lighting and Other Electrical Work

The bridge is lighted by incandescent lamps which are alternated on the two sides and are spaced longitudinally from 100 to 150 ft. apart. From the standpoint of traffic, lights on the bridge seemed even less needed than on the country highways giving access to it. The bridge is straight—horizontally at least—and every reasonable precaution has been taken to prevent vehicles from going over the side. Lights were necessary, however, at each end of the bridge to facilitate the collection of tolls and navigation lights are required by law; the Commission decided, therefore, to light the entire length of the bridge partly for advertising purposes.

Current is received at the New York end of the bridge at 2 300 volts and transformed to voltage suitable for lighting units. Two sets of navigation lights were originally installed—electric and oil—the latter for emergency use only. It was found, however, that the oil lights could not be readily lighted in a high wind, and these have been changed to electric lights operated by a storage battery, to prevent interruption in case of line or station trouble; these lights have proved entirely satisfactory.

Steel lamp posts consisting of 6-in. I-beams bolted to the structural steel fence-posts are used on the truss and viaduct spans. Reinforced concrete

posts are used on the approach spans. The lamps in both cases are supported on structural steel brackets consisting of a single 4-in. car-building channel with flanges turned upward.

Toll Collection and Rates

The facilities for toll collection are on the approach at the New York end of the bridge. The traveled way is here divided, by an island 8 ft. wide, into two roadways, each 20 ft. 6 in. wide. A shelter for toll collectors, with signs giving rate of tolls, is provided on this island. By means of smaller islands, each of these two roadways is again divided, through a small portion of its length, into ways 8 ft. 9 in. wide for the passage of single lines of vehicles. Any or all of these four narrow ways may be shut off by structural steel roadway gates, and each is equipped with a pavement unit, electrically connected to a counter in such a manner that the passage of a vehicle over any one of the pavement units is recorded and a substantially correct traffic count is made automatically. The toll collection area is well lighted and "mushroom" pavement lights and headlight reflecting signals are used where needed. The toll rates are as follows:

Passenger automobile (including passengers)	$1.00
Foot passengers and bicycles	0.25
Motorcycle, or passenger car trailer	0.50
Trucks, tractors, and trailers:	
1-ton, or less, truck or trailer (carrying capacity), or tractor (by weight)	1.00
1-ton to 2½-ton truck or trailer (carrying capacity), or tractor (by weight)	1.25
2½-ton to 5-ton truck or trailer (carrying capacity), or tractor (by weight)	1.50
5-ton truck or trailer (carrying capacity), or tractor (by weight)	2.00
Motor buses:	
Motor bus, seating capacity 16 passengers, or less	$2.00
Motor bus, seating capacity more than 16 passengers	3.00
Horse-drawn vehicles or saddle horses	0.50

Steam rollers, steam shovels, cranes, traction engines, and unusual vehicles of all descriptions, as well as cattle, sheep, and small animals, are permitted on the bridge only upon application to the Toll Collector, and at rates of toll published by the Commission or fixed by the Toll Collector.

A lighted sign is provided at the Vermont end, giving the toll rates, and a guard is stationed there during periods of considerable traffic. A private telephone line connects the two ends of the bridge.

The tolls may be collected at any of four points—the easterly side of the approach, each side of the large island, and the westerly side of the approach, and, at times of heavy travel, collectors are stationed at these four points. When traffic is light, it may be diverted to two lanes, or even to one lane if desired. At each collection point a pedestal is provided upon which a toll register may be placed for the use of the collector in recording tolls. These toll registers record the number of tolls taken by each collector under each of several classifications, and show to the traveler, at time of

registering, the amount of toll paid. Different registers are used for northbound and eastbound traffic. The receipts are taken from the collectors by bank messengers at regular intervals.

At the easterly side of the New York approach is the toll collector's dwelling, in which are located living quarters for the collector and an office from which a view of the bridge to the north and of the New York approach to the south may be had. The switchboard from which all navigation and street lights are controlled is in the office. The dwelling also contains an office for the Commission in which is located the electric traffic counter, enclosed and under lock and key. By means of the roadway gates all vehicles can be diverted, if desired, to the lane immediately adjacent to the dwelling; hence, during the winter and at other times of light travel the toll collector may remain in his dwelling, stepping to the curb to collect tolls when a vehicle approaches.

Total Cost

The total construction cost of the bridge, including foundations, superstructure, approaches, lighting, tollhouses, and equipment, was $967 800. Other items, including interest during construction, administration, legal and engineering charges, and real estate damages brought the grand total up to $1 149 000.

The contract included an allowance by the foundation contractor of $100 per day for failure to complete abutments, viaduct footings, and Piers 3 and 4 on or before October 20, 1928, and the entire work on or before December 1, 1928. It also provided a penalty or bonus of $250 per day for the superstructure contractor for every day lost or saved in the work to or after August 24, 1929, with the provision that the maximum amount thus deducted or added should not exceed $15 000. While certain details, including the final coat of paint on the part of the superstructure below the floor, was not completed by August 24, it was possible to open the bridge to traffic on the date assigned for its dedication, Wednesday, August 26, 1929, and no penalty was imposed upon the contractor.

Acknowledgments

The effect of hydro-static pressure in depositing concrete at considerable depths, as reported in this paper, was studied in a series of tests by Mr. J. Wayne Couter, at the writer's suggestion.

The Merritt-Chapman and Scott Corporation was awarded the contract for construcing the piers. The contract for the superstructure was awarded to the American Bridge Company.

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TOTAL COST

The total construction cost of the bridge, including foundations, superstructure, approaches, lighting, tollhouses, and equipment, was $[illegible]. Other items, including interest during construction, administration, legal and engineering charges, and real estate damages brought the grand total up to $[illegible].

The contract included an allowance to the foundation contractor of $[illegible] per day for failure to complete abutments, viaduct footings, and Piers 3 and 4 on or before October 20, 1928, and the entire work on or before December 1, 1928. It also provided a penalty or bonus of $[illegible] per day for the superstructure contractor for every day lost or saved in the work to or after August 24, 1929, with the provision that the maximum amount thus deducted or added should not exceed $[illegible]. While certain details, including the final coat of paint on the parts of the superstructure below the floor, was not completed by August 24, it was possible to open the bridge to traffic on the day planned for its dedication, Wednesday, August 26, 1929, and no penalty was imposed upon the contractor.

ACKNOWLEDGMENTS

The effect of hydrostatic pressure in depositing concrete at considerable depths, as reported in this paper, was studied in a series of tests by W. J. Wayne Connor, at the writer's suggestion.

The Merritt-Chapman and Scott Corporation was awarded the contract for constructing the piers. The contract for the superstructure was awarded to the American Bridge Company.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

PAPERS

CONSTRUCTION PLANT AND METHODS FOR ERECTING STEEL BRIDGES.[1]

By A. F. Reichmann,[2] M. Am. Soc. C. E.

Synopsis

This paper is concerned with the erection, only, of steel bridges, and, in consequence, classification of the structures will be made more from a standpoint of the problems facing the erection engineer than from type of structure. The physical conditions at the site, time permitted for erection, particular season during which erection is to take place, quantity of river flow, and similar factors have a greater bearing on the way the structure is to be handled than the type of structure itself.

In general, the construction plant for erecting a steel bridge is determined by the type and size of the bridge and the method by which it is to be erected. In consequence, the problems involved by the type and size, which determine the method of erection, will be discussed before the equipment necessary to develop this method of erection, is taken up.

Major Conditions to Be Analyzed

The method of erection depends upon so many conditions that it is impossible to use any set plan of procedure for a given type of structure. Each project should be considered by itself, and the choice which is the simplest and which requires the least equipment is a problem for an experienced erection engineer. In attacking the problem of erection, it might, at first thought, be considered possible to divide the structures into two general classes; that is, railway and highway. However, there is such a wide divergence in both classes of structures, from light single-track railroad structures to the most up-to-date three and four-track railroad bridges, and from the small 20-ft. highway span to the tremendous structures built in the larger cities of the country, that the two classes are interlocking so far as erection problems are concerned. Both highway and railway bridges might be divided into groups as to type of structure. The more important of these types are

[1] Presented at the meeting of the Construction Division, New York, N. Y., January 16. 1930. Written discussion on this paper will be closed in March, 1932, *Proceedings*.

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I-beam spans, girder spans and viaducts, simple truss spans, arches, cantilever spans, continuous spans, suspension spans, and movable bridges. In determining and developing problems of erection and in attempting to arrive at the proper method of procedure, it will be found that the following major conditions will require consideration and analysis:

1.—Type of Structure.

2.—Location of Structure:
- (a) The character of the terrain to be crossed.
- (b) The distance from the center of supply for labor and materials.
- (c) Available space for storage yards and necessary shops.
- (d) Connection with railroad facilities.
- (e) Power and fuel supply.
- (f) Height above the water or ground.

3.—Maintenance of Traffic:
- (a) River.
- (b) Railroad or highway beneath the structure.
- (c) Railroad or highway on the structure proper.

4.—Effects of the Elements:
- (a) Floods and high water.
- (b) High winds.
- (c) Extremely hot or cold weather.

5.—Time Allowed for Construction:
- (a) As provided in the contract.
- (b) As governed by the seasonal effect on the work.
- (c) As governed by schedule of foundation construction.

Type of Structure

In attempting to discuss the various problems listed in the foregoing outline, it will be found that in a great many cases the items are "interlocking" and cannot be treated independently. If the bridge consists entirely of I-beam spans, girder spans, or viaducts, no great consideration need be given to many of the subdivisions following. However, in cases of most other types of structures—especially truss spans that will require a temporary supporting structure—consideration of conditions beneath the latter must be given.

Practically all types will require consideration of the traffic over the bridge, accessibility for delivery of materials, etc. For that reason, no further discussion of the type of structure under that particular heading will be considered, but the various types as affected by conditions under the other following classifications will be mentioned.

Location of Structure

The first consideration under this subject will be the character of the terrain to be crossed. The amount of attention to be given to this problem depends on the type of structure. If the type of bridge is such that no temporary supporting structure is required, little consideration will be necessary. For truss spans of every description, this particular condition will be of major importance. If the structure is over dry land or shallow water, it

will probably be found simplest to use timber falsework as a support; if it is over a deep stream, subject to wide variations in flow, where there is danger of driftwood, scour, and ice, or if there is river traffic to be considered, it may be found impracticable to use much, if any, temporary supporting falsework, and the bridge can best be erected by cantilevering, regardless of whether or not it is the intended permanent type. The erection of light girder spans and viaducts and of suspension bridges does not require major consideration under this heading as, in most cases, the supporting structure is eliminated.

When giving consideration to the problems connected with the distance from center of supply for labor and materials, all types of structures will be involved. If the structure is a railroad bridge, the problems in general entering into the delivery of materials will be simple because only in rare instances are attempts made to erect railroad bridges before the roadbed and track are completed up to the site. In highway bridges, however, the reverse is the case; and the question then develops as to whether it is most economical to build a temporary track to the site, or to arrange for transportation of equipment and materials by the use of trucks or teams. Under this consideration comes also the question of labor supply. If housing conditions are available only at a considerable distance away, it must be determined whether to supply this housing close to the job by the construction of labor camps or to supply transportation for labor to and from the available housing facilities. The latter is of considerable importance in the erection of steel bridges for the reason that the major part of the organization is made up of semi-skilled workers, and to be able to maintain an efficient organization, it must be kept satisfied as to living conditions.

Available space for storage yards, office, and shops will require consideration. In general, it is found that the bridges will be approached by earth fills through low lands, or in the case of bridges in cities, by streets in congested districts. Consequently, it is frequently necessary to arrange for yards, shops, etc., along the fills or streets, and often very little space is available, so that careful planning is necessary to take care of supplies and equipment. Otherwise, it may require going a considerable distance away from the structure for this space, with consequent increased cost of handling and transportation.

The subject of available railroad facilities is to a certain extent connected with the preceding discussion regarding distance from center of supply and conditions at the site and affects this consideration very materially. There may be good railroad facilities at one end of the bridge, but due to the nature of the stream, the method and order of placing the foundations, traffic under the structure, etc., it may be necessary to start the construction of the bridge from the opposite end, in which case it may be a saving to transport the material from the point of delivery on the railroad by hauling, by ferrying in barges, or by transfer on cableways. On some of the larger streams, such as the Ohio River and the Mississippi River, recent developments in transportation have sometimes made it possible to eliminate the railroad problem entirely by delivering the material from the fabricating plants

directly to the site by water. Where this can be readily arranged, it is of considerable advantage in many cases because it puts the material directly under the point where it is to be erected and saves one or more handling charges.

As a rule, the question of available power and fuel supply does not have any great bearing on the method of erection. Its greatest effect will be on the equipment used. If there is electrical power connection, electrically-driven machinery is usually found to be most economical. This condition, however, is rather rare, and usually it is found that power must be supplied by the contractor, in which case, steam-driven or gasoline-driven units can be provided to best advantage. The question of whether steam or gasoline will be selected depends on the available water supply, the facilities for obtaining fuel, and sometimes the type of structure, as, in general, gasoline equipment is somewhat lighter in weight and more easily transported than steam-driven units.

The height of the structure above the water or ground is of major importance and will not only affect the method of erection; it goes even further back than the erection itself, and often affects the design of the structure. When it comes to consideration of the method of erection after the design is definitely decided upon, then this question of height determines whether falsework will be used or whether necessary alterations in the design can be economically made to permit of cantilever erection. As the height of the structure increases, the cost of temporary falsework increases very rapidly, due to extreme wind loads which require additional main supports and much extra bracing. Often, if the structure is low, it is possible to erect truss spans at remote locations more advantageously and then float them on barges to their final location. This will especially merit consideration if the structure is close to the surface of the water, and the water is excessively deep at that point.

Maintenance of Traffic

Under this heading, the maintenance of river traffic is of first importance for the reason that traffic on navigable streams in the United States is controlled by the Federal Government, and in case of interference thereto, permission from the United States War Department is necessary. In many cases there is only one available channel for river traffic, and, therefore, no supporting structure can be placed in the river at that point. As a consequence, it may be necessary to arrange for the erection of the span by cantilevering or by floating the structure in place, or by some such means although other considerations would indicate that falsework would be most economical. In the northern part of this country it is sometimes possible to arrange the time of erection, so that the setting of steel over channels can be prosecuted when navigation is closed by ice. In any cases where it becomes absolutely necessary to do any construction that in any way affects river traffic, arrangements must be started for this well in advance of the time it is needed because the Federal Government, in granting permission to do this, will require various transporting companies to be notified and

warned of any obstruction to navigation that may be contemplated, and, in addition, notices must be published in local newspapers.

In the case of railroad or highway traffic beneath the structure, it is usually possible to restrict this traffic partly, by narrowing the lanes of traffic, so that they can be crossed without great difficulty. If this cannot be done, it may be possible to detour the traffic temporarily, so that this problem is not of as great moment as the maintenance of river traffic.

The maintenance of traffic over the structure may be very important, as in the case of replacing an old span. In a problem of this kind, the maintenance of traffic will have a great bearing on the method of erection. In some cases it will be found possible to carry the traffic on the old structure while building the new one around it, or it may be possible to place temporary supporting falsework and carry traffic on this falsework at the same time a new structure is carried on it. It does not have as great effect on the method of erection as it does on the cost. Where there is any traffic to be maintained, it will have considerable effect on the cost of the work for the reason that it cannot fail to cause many interruptions, and consideration should be given to a method which will involve the least interruption to the progress of the work by traffic. The maintenance of traffic over the structure is usually more easily arranged than that under the structure because, in arranging this traffic, the erector is dealing with parties vitally concerned with the completion of the structure, and, in consequence, can more easily arrange for the necessary interruptions in traffic.

Effects of the Elements

The most important considerations under this heading are floods and high water. In erecting bridges over most of the major streams of the United States, it will be found that it is of most importance to place temporary supporting structure during the periods when low water may be expected. Therefore, from necessity, the entire program will be predicated on the time when the water will be low. Even if the fluctuation between high and low water is small, the periods of high water will bring down driftwood or floating ice, or it will cause scour in the river bottom that may prove disastrous.

A high wind must be anticipated at any time and provision must be made for it. This means consideration of extra bracing and guys to the structure. There are localities in the United States where, at certain times of the year, high winds are especially prevalent, which may affect the time when the work is to be prosecuted. This requires consideration not only from a standpoint of safety, but also from a standpoint of economy. However, there may be long periods when it is impossible for men to work on a high structure due to the wind.

There are few localities in the United States where extremes of hot or cold weather will be a great problem, except as it affects the cost of the structure. It will not be necessary to suspend work entirely on account of hot or cold weather, but progress may be seriously impaired, with consequent increase in cost.

Time Allowed for Construction

The time allowed to complete the work, as provided in the contract, may be of vital importance in determining how it is to be done. It may be advisable from a standpoint of economy to erect the structure from one end, but the time allowed may require that the work be prosecuted from both ends. It is often necessary, therefore, to decide whether to work from one end of the structure only, and work long hours with consequent extra expense for overtime; to work in night shifts, with consequent great reduction in efficiency; or to work from both ends of the structure, with consequent increase in force, equipment, and materials. Often the date of completion of the contract will require the bridge builder to prosecute the work at a time when it is extremely hazardous on account of risks from high water, ice, etc.

There are certain seasons of the year when one can almost be assured that he must contend with ice, driftwood, current, scour, etc., and it is, therefore, often necessary to arrange and schedule the work so that certain parts of it, which may be seriously affected by these contingencies, can be prosecuted at a time to avoid undue risks. In order to meet this condition on even a small portion of the work, the schedule of the entire project is often affected. The same problems that face the steel erector in connection with river conditions very often affect the foundation contractor, so that he may be unable to alter his foundation schedule and it will be imperative, therefore, for the steel erector to adapt his schedule of erection to fit that of the foundation.

From the foregoing comments, it will readily be seen that all these contingencies have their effect on the method by which the work is to be done, and must be considered both singly and in combination in properly laying out the program of erection.

Equipment and Plant

The problems which have been discussed herein in connection with the method of prosecuting the erection will affect the amount and character of the equipment necessary. It is apparent, therefore, that before deciding definitely upon these factors, the method by which the work is to be prosecuted must be determined. Consequently, it is out of the question in the scope of this paper, to discuss the amount and type of equipment required for any particular kind of structure; therefore, only a general description of the types of equipment used for bridge erection will be given. Almost every variety of power equipment is used in some form or other for the different types of steel erection. Only the more important of such equipment will be described.

Locomotive Erecting Cranes.—Probably the most universal type of equipment used for erecting bridges is the locomotive crane (see Fig. 1). This is an adaptation of the machines originally designed for wrecking cranes for use by the railroads. Originally, the railroad wrecking cranes were used, the only change being to replace the short, curved boom with a long, straight

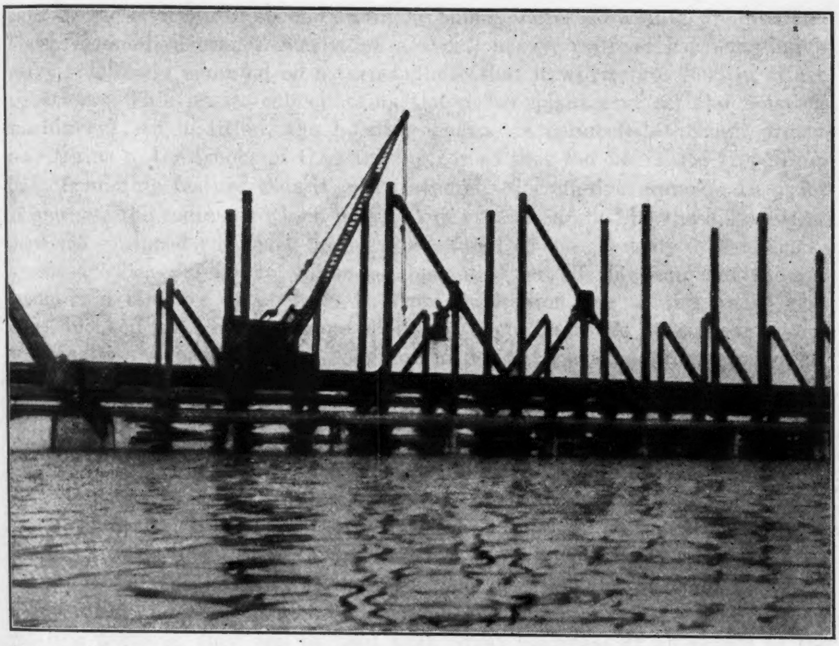

FIG. 1.—LOCOMOTIVE CRANE ERECTING BOTTOM CHORD AND WEB SYSTEM ON FALSEWORK DERRICK CAR SETTING PLATE GIRDER ON BENT.

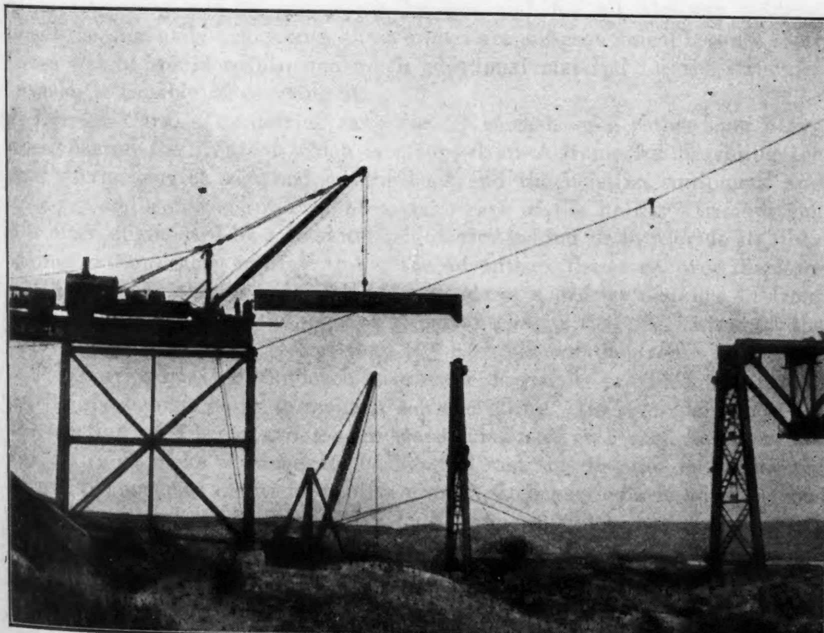

FIG. 2.—DERRICK CAR SETTING PLATE GIRDER ON BENT.

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boom. Later, these cranes have been developed more specifically for erection work by the addition of special auxiliary hoists, increased counterweights, etc. These locomotive cranes consist of a short, heavy, railroad car carrying a heavy steel body mounted on a turn-table so that it will rotate 360° in either direction. This heavy cab contains the power plant and all the hoisting machinery. In addition, the hoisting engine is connected through proper gear trains to the trucks of the carrying car so that the car is self-propelling. It is from this feature that it gets its name, "locomotive crane". In order to increase the radius at which heavy loads can be handled by these machines, they are equipped with jack-beams at each end of the car body. These jack-beams are heavy I-beams which can be pulled out of the frame of the car (usually a distance of about 8 ft. from the center line of the track) and when blocked up, give the crane a base of about 18 by 24 ft. These cranes are usually also provided with booms so made that when handling excessively heavy lifts a short boom is used, while for setting lighter loads at a greater height above the track, additional sections can be added.

Due to the fact that these locomotive cranes—in order to have stability for handling loads at any advantageous distance from their center of rotation—must be excessively heavy, their use on bridge erection is confined to those structures which, by their design and method of construction, will support heavy loads. In the case of I-beam spans, girder bridges, and viaducts within their capacity, and truss spans the dead weight of which requires heavy falsework, these machines make ideal rigs for erecting. Due to their rotating features, they can be used with equal readiness at all points of the compass from where they are located. In most cases of truss spans erected by cantilevering and of suspension bridges and for all highway bridges, except the unusually heavy ones, these cranes are not economical because their excess weight would require too much additional material for the structure to make it capable of carrying it.

Derrick Cars.—The derrick car (Fig. 2) consists of a rather long heavy frame car on the front of which is mounted an A-frame for providing the mast of the derrick rig, and on the back end the hoisting equipment and sufficient additional counterweight to take care of the uplift. In addition, it is often augmented by a "counterweight car" behind it, to provide stability against extraordinary uplift. It has the advantage, therefore, over the locomotive crane in that it will pick heavy loads at a greater distance in front of the car; but it has the decided disadvantage in that the swing of the boom is limited to slightly less than 90° each side of the front.

The derrick car, like the locomotive crane, is usually provided with proper driving mechanism so as to make it self-propelling. Its principal use is for the erection of I-beam and girder spans, viaducts, etc., the character and size of which make the loads to be handled and the distance to be reached beyond the capacity of the locomotive crane. It is particularly advantageous for setting heavy girder spans 80 ft. long, or more, which, under present-day railroad traffic, often run from 75 to 100 tons in weight.

Derricks.—These are stiff-legs or guy derricks made in various designs and capacities. The difference consists in the bracing of the mast. If there

are two stiff legs, the derrick is a "stiff-leg derrick"; if braced with guy ropes, it is a "guy derrick". A combination derrick has one stiff leg braced with guy ropes at right angles.

Derricks can be used for practically every type of structure, but as they are usually difficult to move from place to place, they are not often used except for the erection of small isolated structures, or for handling the steel from cars to storage at the storage yard. They are, however, often of distinct advantage for the erection of bascules which are to be built in open position and for the towers of vertical lift bridges. In both cases, this amounts to erecting a truss span standing on end, and, consequently, the structure is not spread over a great deal of ground and is readily reached without moving the equipment. Derricks are often mounted on barges forming derrick boats which are particularly serviceable for the placing of temporary supporting structures for bridges, such as driving piles, setting falsework, etc., and if the permanent structure itself is not too high above the water, they can be used to advantage in erecting the steel.

Travelers.—Briefly, travelers may be described as derricks with their hoisting equipment mounted on a movable platform. They are of almost every conceivable type and are usually designed and built to fit the requirements of each individual structure. Depending on the particular type of structure for which they are designed, they may be operated on the floor of the bridge or span from one truss to the other on the top chords. If they are operated on the floor of the bridge, some special provision usually has to be made to move the material either through them or around them in order to get it within reach of the booms. They are of particular advantage when the material is to be delivered to the site by water; therefore, the question of getting the material through or around them does not enter into the problem. They have a further advantage in that as they operate over the floor of the bridge, there is no special provision necessary to get them from one span to the other as the erection progresses over a series of spans. This is offset by the disadvantage of having them sufficiently strong and heavy to carry the long boom necessary to reach to the upper parts of the structure.

If the travelers are operated on the top chord of the structure (Fig. 3), they must be constructed to span from one chord to the other and require the necessary special construction on the trusses to permit moving them from one span to the next. Offsetting this they have the advantage, since they are entirely above the steel to be erected, that the booms may be much shorter, permitting lighter construction. Furthermore, placing them on the top chords leaves the floor system entirely unobstructed for bringing forward the materials required.

Travelers are used for the erection of almost every type of bridge (see Fig. 4). In cases where truss bridges—either due to their type or to some of the conditions heretofore discussed—are to be erected by cantilevering, some type of traveler is ordinarily used because less weight is involved for their capacity than with cranes or derrick cars. They can be modified in innumerable ways to adapt them to special needs. In the case of towers for suspension spans and extra long bascule spans to be erected in the open

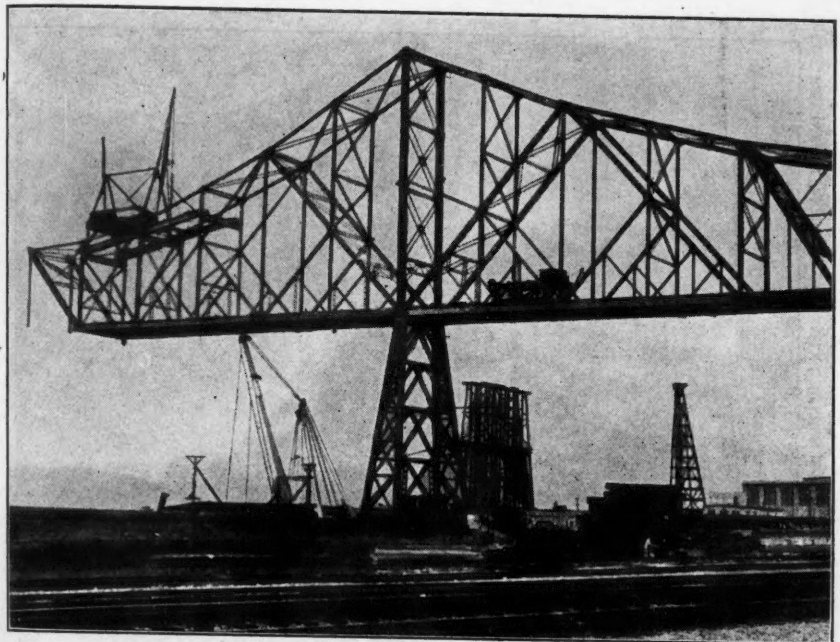

FIG. 3.—TOP CHORD TWO-BOOM TRAVELER, CANTILEVER ERECTION.

FIG. 4.—JINNIWINK TRAVELER ERECTING GIRDER SPANS ON PIERS.

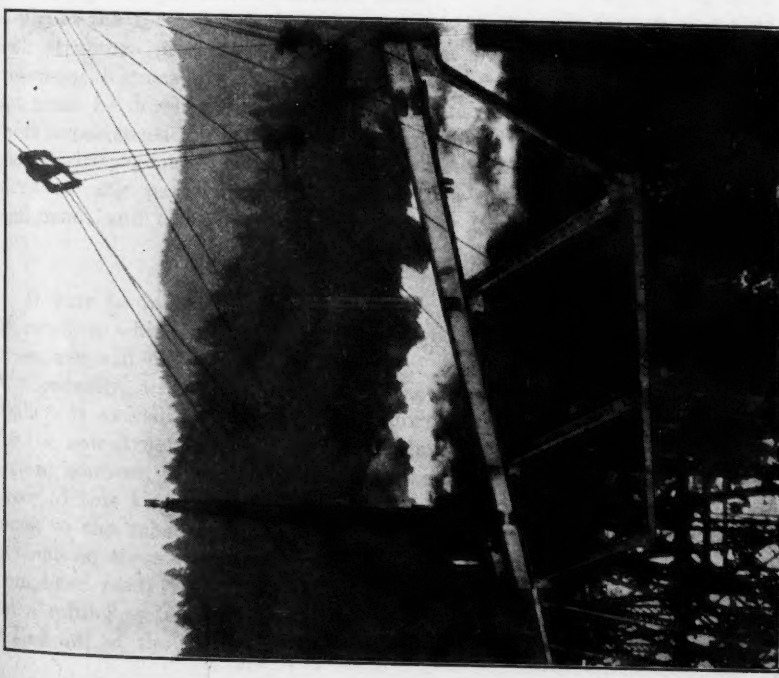

FIG. 5.—ERECTING TRUSS WITH CABLEWAYS; LOCOMOTIVE CRANE IN BACKGROUND SUPPORTING ONE END OF TRUSS.

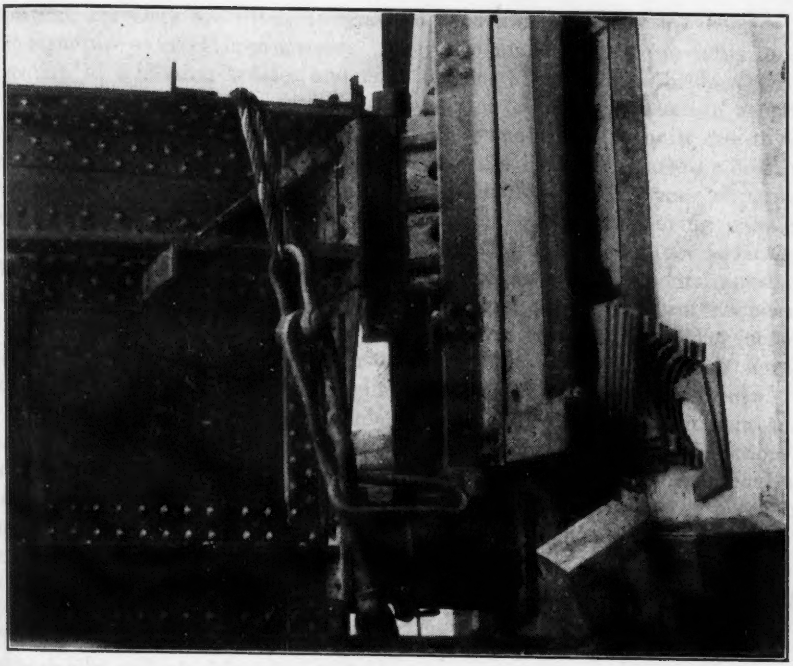

FIG. 6.—HYDRAULIC JACK RAISING SPAN TO RELEASE FALSEWORK.

FIG. 4.—HYDRAULIC JACK RAISING SPAN TO RECEIVE FALSEWORK.

FIG. 5.—ERECTING TRUSS WITH CABLEWAYS; LOCOMOTIVE CRANE IS BACKGROUND SUPPORTING ONE END OF TRUSS.

position, travelers are often designed to be drawn vertically up the side of the structure as erection progresses. They also adapt themselves readily to the erection of stiffening trusses and floor systems of suspension bridges.

Cableways.—Often the erector is confronted with the problem of keeping his erection rig entirely independent of the structure to be built due to the character of the structure. When cantilevering light structures or erecting arch bridges which are cantilevered until finally erected, these structures are often too light to carry the stresses from equipment during erection. In this case an overhead cableway is often found to be a distinct advantage. This is especially applicable in cases of bridges crossing deep ravines where hillsides, adjacent to each end of the structure, provide natural anchorage for the cableways (Fig. 5). They scarcely need description as they consist of a trolley provided with proper hoisting falls and operating on a rope stretched across the stream or ravine above the site of the new bridge.

A description of the vast number of special tools and equipment used in bridge construction will not be undertaken in this paper. Erection of steel structures has kept apace with the development of this mechanical age and where, years ago, a tremendous amount of hand work was necessary, practically all operations are now done by mechanical appliances. For example, rivets are now almost universally driven by air or electricity; and the oxy-acetylene cutting and welding flame and the electric arc are used for the removal of old structures and for alterations to the new in the field. Timber for falsework and for the decks of bridges is framed by the use of electrically and air-driven portable tools.

From the foregoing it is apparent that in the erection of steel bridges, each structure presents a distinct problem which requires its own special treatment. Procedure very rarely can be governed entirely by precedent, but must be developed by a study of its individual characteristics by engineers experienced in these lines. Consequently, the foregoing discussion has been entirely general, and no attempt has been made to describe the development of any particular type of structure or to attempt to outline the equipment and tools necessary for any one type.

Unusual Methods of Erection

It may be interesting to touch briefly on a few of the unusual methods of erection which are sometimes employed when the ordinary methods of procedure will not apply.

Generally, when the traffic over the structure which it is planned to replace is exceedingly heavy, every effort is made to change the alignment for the new structure so that it may be built without interference with traffic. Often, however, physical characteristics will make this impossible. In some cases of this kind, arrangements have been made to build temporary extensions to the substructure and then to construct the new bridge beside the old one on these temporary extensions. After the new structure is entirely completed ready for traffic, both the old and the new structures are placed on a rolling system and within a comparatively short time, the old span is rolled out of the way and the new structure moved into its final position

ready for service. To handle a structure in this way requires—in addition to the rolling system—some means of raising the structure to release the rollers. For this purpose, hydraulic jacks (Fig. 6) have been developed with capacities as high as 2 500 tons per jack. Water is generally used to operate them except during periods of low temperature when mixtures of alcohol in water or even pure alcohol are used.

In the erection of permanent cantilever spans, it is usually possible to erect the suspended span by temporarily cantilevering one-half of it from each permanent cantilever arm. Occasionally, however, the stresses developed by this method make it impractical to extend the cantilever beyond the permanent cantilever arm. In a few cases, therefore, the suspended span has been erected at a remote location, transported on barges to the site, and raised into final position. In some cases the span has been raised by the use of hydraulic jacks; in others, where the height was considerable, the suspended span has been raised by the use of cables running over sheaves mounted on the ends of the cantilever arms. To reduce the power required to lift the span, counterweights are used to balance the lifted span.

The erector will sometimes be confronted with the problem of constructing a single truss span, where, because of exceedingly deep water, swift current, or because of navigation, it is an impossibility to place any supporting structure under the span. In such cases it has been found necessary to erect temporary anchor arms at each end of the structure in order to make a temporary cantilever of this simple truss span. Such a method requiring, as it does, the erection and later removal of a large quantity of material, is, of course, expensive and is only resorted to where no other means are available.

Concluding Remarks

The more important problems of erection are only briefly discussed in this paper, because there is practically no limit to the special cases of erection and equipment that will confront the engineer when attacking an individual problem. It is hoped, however, that sufficient ideas have been advanced to serve as a general guide in the solution of specific problems.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
PAPERS

FULTON STREET, EAST RIVER TUNNELS, NEW YORK, N. Y.[1]

BY MILES I. KILLMER,[2] M. AM. SOC. C. E.

SYNOPSIS

This paper treats the specific problems encountered by the contractors in constructing the two tubes of the Fulton Street, East River Tunnels, in New York, N. Y. Useful information is given concerning the size and general types of equipment necessary and the manner of using it to the best advantage. The paper supplies a general picture of the contractor's power house, required on a job of this scope, and the assembly and erection methods of various phases of the work, especially the shields, are described in detail. The paper closes with an interesting account of construction problems met during the building of the High Street Station.

INTRODUCTION

This paper gives a description of the plant used on the Fulton Street Tunnel, which is situated on what is known as Route 101, Section 1, of the new subway of the City of New York. The contract began at Church Street about 100 ft. south of Park Place in Manhattan (see Fig. 1). Local and express tracks run south on Church Street to Fulton Street, where the local tracks terminate at a station next to the Hudson Terminal Building. The two express tracks turn east, pass under the southbound tube of the Brooklyn-Manhattan Transit (B. M. T.) Company in a short section of concrete tunnel, and continue east in separate cast-iron tubes to the end of the contract at Jay and Nassau Streets, in Brooklyn, N. Y. Following the cast-iron tubes from west to east, one passes under the northbound "B. M. T." tube, St. Paul's Churchyard, Fulton Street, the East River, Cranberry Street, High Street, and under private property to the end of the job.

The tubes were constructed by the shield method under compressed air. The working shafts were at Fulton and Front Streets, Manhattan, and on Furman Street, Brooklyn. Shields started east and west from each of these shafts in each tunnel, making eight shields in all. From January until May,

[1] Presented at the meeting of the Construction Division, New York, N. Y., January 16, 1930. Written discussion on this paper will be closed in March, 1932, *Proceedings*.

[2] Mgr., Mason & Hanger Co., Inc., New York, N. Y.

1929, all eight of the shields were under way together. In Brooklyn, the ground pierced by the shields was mostly sand, gravel, and boulders. In Manhattan, it was a fine sand with clay admixture and was very favorable for tunneling. The Manhattan shields passed under the tracks of the Interborough Rapid Transit (I. R. T.) Company, in William Street and in Broad-

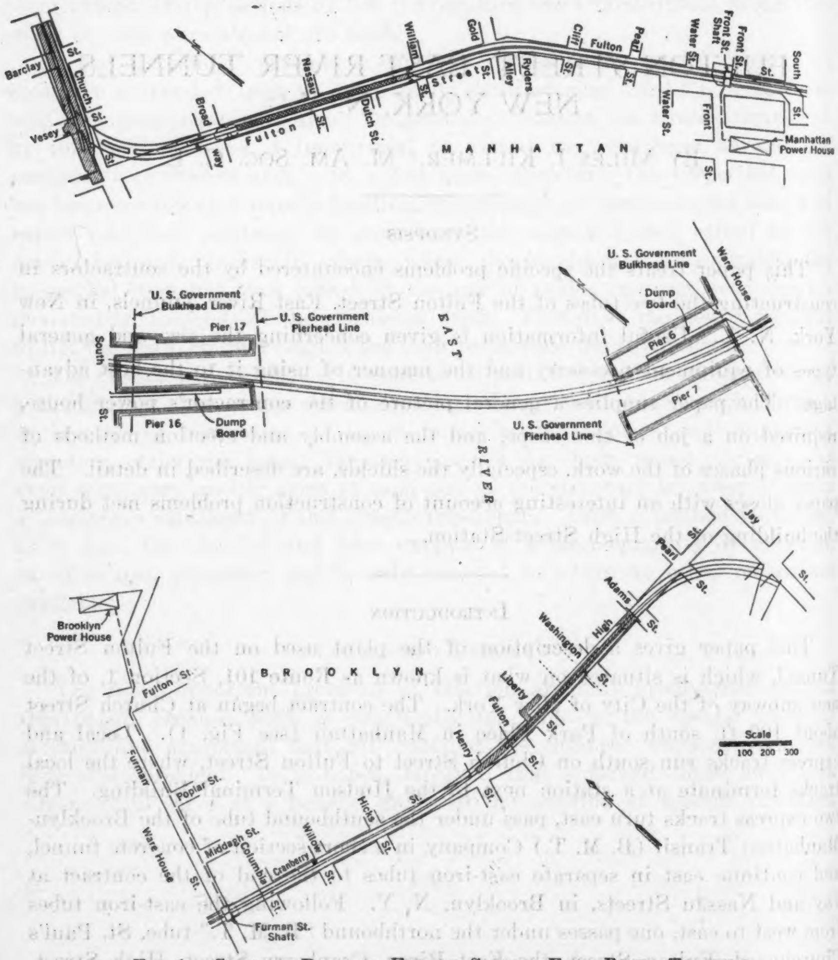

FIG. 1.—GENERAL PLAN OF FULTON STREET, EAST RIVER TUNNELS.

way, and the easterly "B. M. T." tube near Church Street; and the Brooklyn shields passed under the east approach to the Brooklyn Bridge. Along "the line of march" through Fulton Street were many tall buildings the footings of which were close over the shield, vertically, and only 2 or 3 ft. away from the outside of the tunnel, horizontally. These buildings were not underpinned. The air pressure used in driving the shields and in putting down the mid-river sumps ranged all the way from normal to 48 lb. per sq. in.

In addition to the foregoing, the work included the construction of two stations in Manhattan, by cut-and-cover methods, and a station in Brooklyn by tunneling.

The contract was awarded in November, 1927. The two shafts, the open-cut work on Church Street, and the station on Fulton Street, were built by sub-contractors. This paper describes only the plant used on the remainder of the contract.

Power Houses

The power house on a compressed-air tunnel job is one of the vitally important items of plant. On the reliability of the machinery depends the safety and even the lives of the men working in the heading; on its adequacy depends in large measure the economy of the operation. There were two power houses for this job, one at each end. The specifications called for a low-air capacity of 28 000 cu. ft. of free air per min. for each plant. The two plants as built were almost alike. The low-air, high-air, and hydraulic installations were identical, but the electrical installations were necessarily different. However, the work was no sooner under way than it became apparent that the demand for low air on the Brooklyn side, because of the loose character of the ground, was going to be much greater than in Manhattan. This condition persisted throughout the duration of the job. The Manhattan machines were seldom used to capacity, but all the Brooklyn machines were almost continuously being pushed to their limit, and even then there were times when it was not possible to keep up the air pressure to what was desired in the river headings.

All the machinery was operated by electricity. In Manhattan, the current was received at the power house at 13 200 volts from one feeder and at 7 500 volts from the other two feeders, and it was stepped down to 2 200 volts for the larger motors and to 220 volts for smaller power demands. In Brooklyn, the current was received at 4 300 volts and used at that voltage for the large machines; it was stepped down to lower voltage for the small motors.

Power was supplied by two commercial companies as sub-contractors. The electric current reached each power house by three entirely independent feeder lines, coming from three different sub-stations. This precaution was necessary, of course, to insure power in the event of a break-down or accident along one of the feeders. The power-house engineer and electrician always had to be on the alert. In a number of instances, break-downs did occur and all machines came to a stop. It usually took from 3 to 5 min. to start the compressors again.

In each power house there were five low-air machines of 5 600 cu. ft. capacity. Four in each plant were new, and one was a used machine. The new machines were 29 by 29 by 21 in., and were operated by synchronous motors of 625 h. p., for 2 200-volt, 3-phase, 60-cycle current with a full-load speed of 180 rev. per min. The motors on the Brooklyn machines operated at 4 300 volts. The used machines were 28 by 28 by 24 in., with synchronous motors of 600 h. p., at 164 rev. per min. In the new machines the after-cooler was of a horizontal type, while in the old ones it was vertical. The old

machines had hand-operated valves, and the new ones had valves designed to cut in when the receiver pressure fell below a certain set amount. These automatic valves depended on the receiver pressure as a source of power to open and they gave some trouble at first. There was a small blow in one of the headings when one of the shields was being started from the Front Street Shaft. The receiver pressure fell so low that the valves failed to operate just when they were most needed. The old machine with hand-operated valves was started and sufficient air was obtained from it to meet the emergency. After this incident, all the automatic valves were connected to a separate receiver which was kept supplied with the necessary air at the desired pressure by one of the high-air machines.

The compressors gave admirable service; the operators, as well, proved their mettle; dramatically, on one occasion, when the Brooklyn power house was struck by lightning and less conspicuously through many months of unfailing service. In Brooklyn, there were weeks and weeks when all five machines were compressing air at maximum load, 24 hours per day and 7 days per week.

The air for the compressors was taken in above the roof. In Manhattan, the inlets were equipped with filters to remove some of the dust of the city streets. By taking in the cooler air from outside the power house, one not only secures better air, but better compression economy.

The demand for high-pressure air is usually large on a tunnel job. Each power house had three 20 by $12\frac{1}{2}$ by 14-in. machines of 1 300 cu. ft. capacity. The motors were synchronous and rated at 225-h. p., 220-volts, 3-phase, 60-cycle, 257 rev. per min. The high-pressure air could be turned into the low-air lines if need be and was, in fact, so used in Brooklyn on several occasions.

The hydraulic machinery in the power house consisted of three four-plunger horizontal pumps; the plungers were $1\frac{3}{4}$ in. in diameter with 12-in. stroke. The pumps operated at 50 rev. per min., had a capacity of 24 gal. of water per min., and were driven by 100-h. p., gear-connected, 220-volt, 3-phase, 60-cycle motors, rated at 700 rev. per min.

The accumulator was of the hydro-pneumatic type in which the load was applied by a cylinder charged with compressed air instead of by the old weight method. The size was 8 by 42 in.

The power houses were connected to the shafts by the necessary air, water, electric, back-pressure, and telephone lines, laid in trenches below the street level. In Manhattan, the power house was only one block from the shaft, and two 14-in. lines sufficed to carry the low-pressure air. In Brooklyn, the distance was 1 600 ft., and two 16-in. lines were used. The contract required two lines to each heading. The various air lines are cross-connected, in order to secure maximum safety and flexibility.

Fig. 2 shows the manifolds and valves at the top of the shaft in Manhattan. A shanty was built on this spot for the gauge tenders, and back-pressure lines were returned from all headings to this shanty, to the power house, to the engineers' office, and to the superintendents' office.

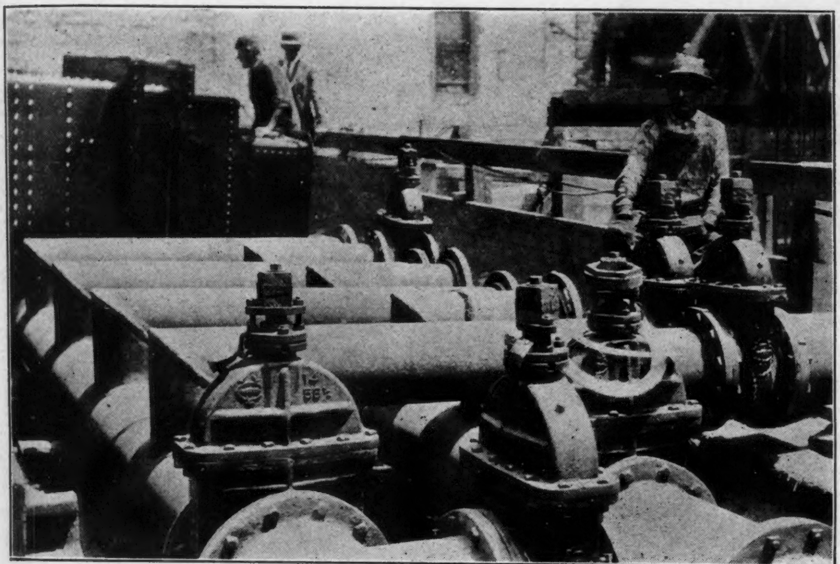

FIG. 2.—MANIFOLDS AND VALVES AT TOP SHAFT IN MANHATTAN.

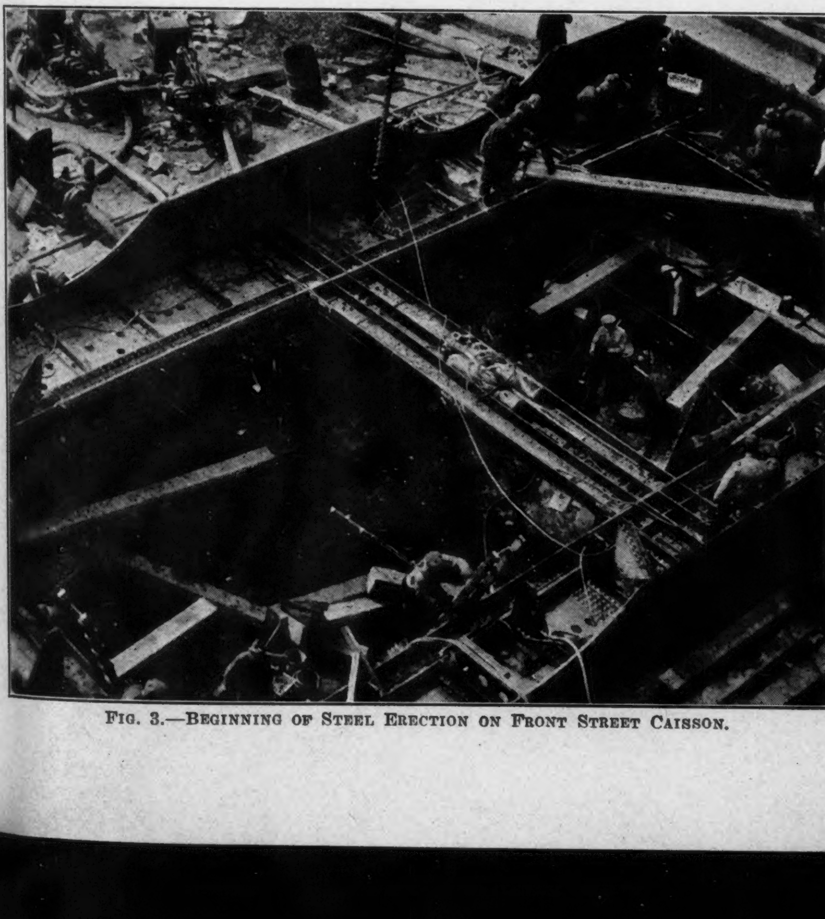

FIG. 3.—BEGINNING OF STEEL ERECTION ON FRONT STREET CAISSON.

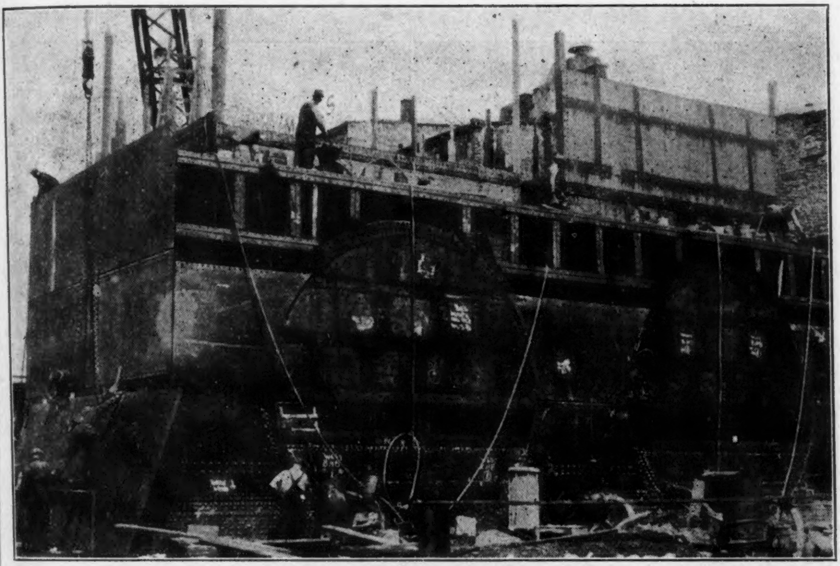

FIG. 4.—STEEL ERECTION AT LATER STAGE.

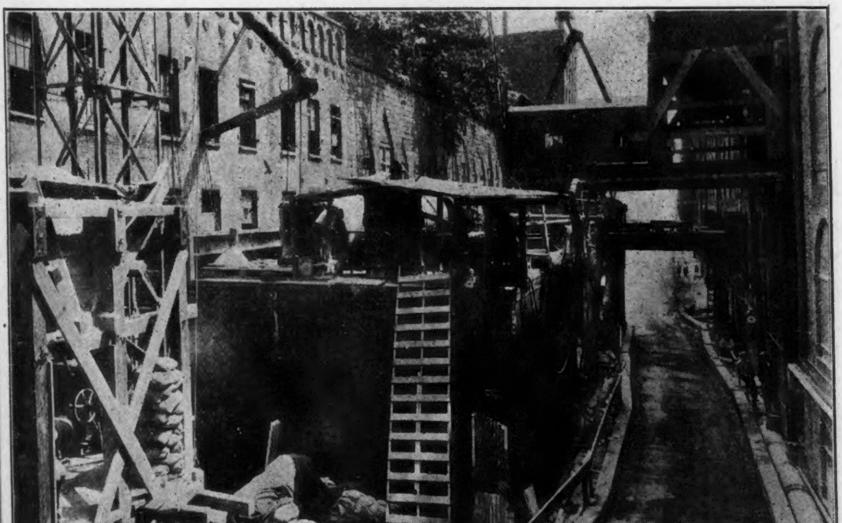

FIG. 5.—SINKING FURMAN STREET SHAFT.

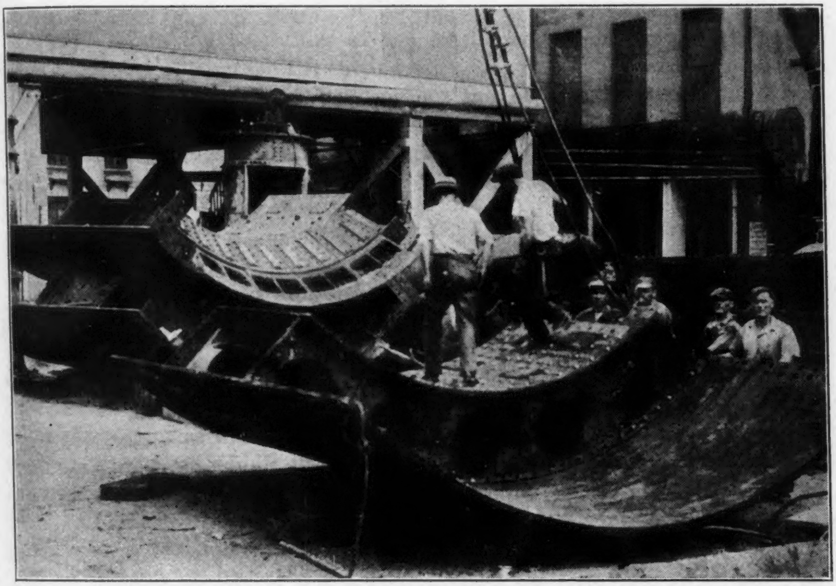

FIG. 6.—TWO SECTIONS OF SHIELD.

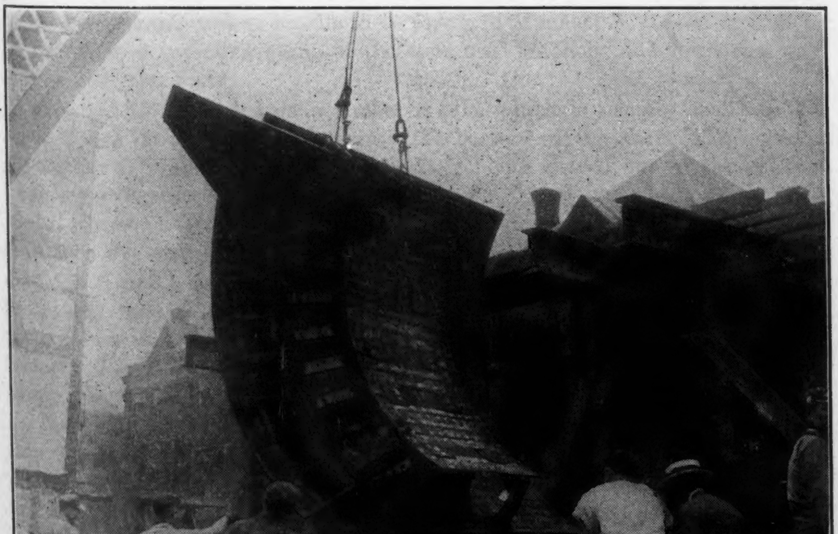

FIG. 7.—ONE OF LOWER-QUARTER SECTIONS OF A SHIELD IN THE SLINGS OF A HEAVY DERRICK, READY TO BE LOWERED.

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Caissons

Fig. 3 shows the beginning of the steel erection on the Front Street caisson. In the photograph the cutting-edge steel has been placed in a shallow excavation and the work of building up the hollow steel walls has been started. Fig. 4 is another view showing the steel erection further advanced. The circular openings (closed by bulkheads) through which the shield leaves the caisson can be plainly seen. Fig. 5 shows the sinking of the shaft on Furman Street, in Brooklyn.

Shields

The contract required the construction of eight tunnel shields. Since the material to be encountered in all the tunnels was sand or gravel and, since there was no possibility of "shoving blind" at any stage of the job, the shields were entirely open, and there were no unusual features in their design. One platform extended across the horizontal diameter. The spaces above and below the platforms were each divided into three pockets by two vertical partitions. The lower middle pocket was made large enough to permit a muck car to pass through to the face, but the conditions of the job never required the use of this facility. The skin-plates of the shield consisted of two $1\frac{1}{4}$-in. plates, riveted and welded together.

Two circular girders or diaphragms formed the stiffening structure. As much as possible of the work of fabrication was done at the shop. A quadrant of the skin and quadrants of the stiffener girders with cutting-edge or hood castings attached, were riveted together in the shop. Fig. 6 shows two of these sections. The largest of them weighed about 20 tons. Part of the boom of the heavy crane used to handle these pieces in Manhattan, is seen in Fig. 7. In Brooklyn, a traveling crane of the type used in shops and foundries was erected over the shaft.

Each shield is erected on a timber cradle, which is usually built on the ground, and is lowered complete into the bottom of the shaft. The cradle also serves to support the shield while it is being shoved out of the caisson when the tunneling operation is begun.

An ingenious device was used to speed up shield erection. Rollers were built into the cradle so that the assembled shield could be rotated about its axis. The seams along which the skin-plates were to be welded could thus be brought "into the clear" for easier access. The insertion of the jacks into their places was made much simpler, because they were all put in at the bottom. The shield was rolled by the derrick which pulled on a wire cable wrapped several times around the outside.

The outside diameter of the shield was 18 ft. 10 in., the thickness of the skin was $2\frac{1}{2}$ in., and the outside diameter of the tunnel, 18 ft. 3 in., making the theoretical clearance between the outside of the iron and the inside of the tail 1 in., radially. This clearance served very well in the course of the job. In Manhattan, two shields went around curves of 335-ft. radius without the tail binding unduly against the cast-iron lining. In going around this sharp curve, special efforts had to be made to build the iron so as to overcome the

tendency of the tail of the shield to shorten the horizontal diameter. The tail was long enough to lap over two rings and 6 in. on the third ring. This is the minimum length of tail that will permit the removal of a broken segment in a ring after a shove has been completed.

The shields were very heavily constructed and stood up well for the duration of the job. On the Brooklyn side, boulders were of very common occurrence; in some cases, pieces were broken out of the cast-steel hood segments by hidden boulders. Both land and river shields were equipped with hoods extending 2 ft. 10 in. farther ahead than the cutting-edge; the hoods on the land shields reached to 1 ft. below the springing line on either side, while those on the river shields reached only to a point 1 ft. above the springing line. The total length of the shield along the bottom was 15 ft., of which 6 ft. 6 in. was in the tail, 5 ft. 2 in. was between the rear diaphragm and the front diaphragm, and the remainder made up the forward part of the shield consisting of the cutting-edge castings and the steel structure to which they were anchored. Fig. 8 shows the dimensions in the cross-section of a river shield.

Hydraulic Equipment.—The hydraulic equipment on each shield consisted of shoving jacks, table jacks, and the erector. The shoving jacks were 10 in. in diameter and were equipped to be pushed out or pulled back by hydraulic power. With a pressure of 6 000 lb. per sq. in., the eighteen jacks were capable of exerting a forward thrust of 4 000 tons. The shoving jacks had the usual offset foot so that the pressures exerted by them were transmitted to the web of the cast-iron segment and not against the flange. In spite of this design, there were many instances in which tunnel segments were broken by the jacks during a "shove." To reduce the number of such breakages, steel shoes were made in the blacksmith shop and fastened to each jack. These shoes were designed to limit the area over which the thrust was exerted against the segment, and also to overcome the tendency of the moving part of the jack to twist.

Of course, each jack had its own control spindles by which it could be pushed out, exhausted, or pulled back. There were eighteen jacks on each shield, eight above the springing line and ten below. This arrangement proved satisfactory in controlling the moving of the shield.

The jacks stood up well throughout the course of the job and there was only a small amount of repacking required to keep them all in working order. This work was done on Sundays and it can be stated that throughout the entire job, every shield averaged 90% of its rams in working order.

Three tables were provided in the horizontal platform, one in each of the three pockets. They were of exceptionally heavy construction and lasted in the land headings for the duration of the job. In the river headings, they were put out of commission after the shields had passed the river bulkheads.

The erector was mounted at the edge of the horizontal platform of the shield on the axis of the tunnel. The motion of rotation (extending to about 380°) was effected by a pinion on the erector shaft, moved by a rack above it just under the operator's platform. The erector arm was extended or withdrawn by a hydraulic cylinder within the erector itself. Fig. 9 is a view

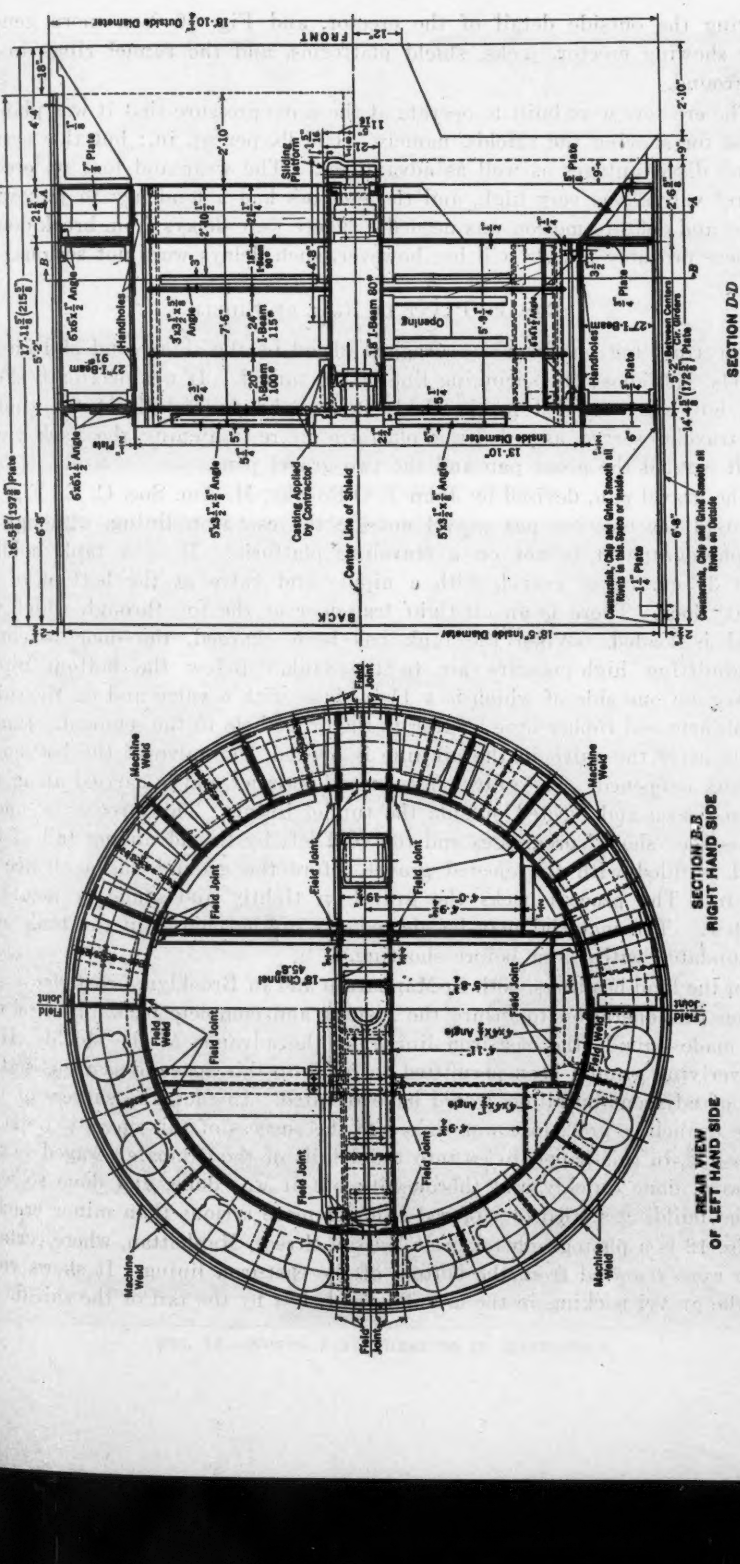

FIG. 8.—TYPICAL CROSS-SECTION OF RIVER SHIELD.

showing the outside detail of the erector, and Fig. 10 is a more general view showing erector, jacks, shield platforms, and the tunnel rings in the foreground.

The erectors were built to operate at the same pressure that it was planned to use for shoving the shields, namely, 5 000 lb. per sq. in.; but this system offered disadvantages as well as advantages. The wear and tear on erector control valves was very high, and the erectors had a tendency to jerk when a slow and definite motion was needed. There were delays from break-downs, as there probably always will be; however, such delays were not serious.

Tunnel Plant in Rear of Shield

The traveling platform was dragged ahead by the shield and slid on the tie-rods, just above the springing line of the tunnel. It was originally 20 ft. long, but was extended in the field; it was the full width of the tunnel. The traveler served as a bolting platform in re-tightening the back rings, and it carried the grout pan and the two gravel pans.

The gravel pan, devised by John J. O'Rourke, M. Am. Soc. C. E., Fig. 11, was used for ejecting pea gravel outside the cast-iron lining, although in the photograph it is not on a traveling platform. It is a tank holding about 33 cu. ft. of gravel, with a nipple and valve at the bottom of its conical floor. There is an air-tight trap-door at the top through which the gravel is loaded. When the tank has been charged, the door is closed by admitting high-pressure air to the tank. Below the bottom nipple is a tee, on one side of which is a $1\frac{1}{2}$-in. hose with a valve and on the other side an armored rubber hose leading to the grout hole in the segment. Immediately after the valve in the air line is opened, the valve at the bottom of the tank is opened. The gravel falls into the air jet and is carried along the armored hose and ejected outside the tunnel lining. This process is under way as the "shove" progresses and the void left by the advancing tail of the shield is filled with the ejected gravel before the ground has a chance to cave in. The process packs the gravel in tightly and almost cements it together. The material may be ejected dry or the charge in the tank may be inundated with water before shooting.

On the land headings, both in Manhattan and in Brooklyn, great stress was laid on this operation to insure the instant and complete back-filling of the void made outside the cast-iron lining by the advance of the shield. Had the overlying ground been permitted to fall into this space, disastrous settlement of adjacent buildings would have resulted. In fact, the success of the entire tunneling procedure was linked to the success of this operation. It is impossible, in this paper, to go into the details of the campaign waged to get this work done quickly and thoroughly; but it was done, and done so well that no building settled or suffered injuries more serious than minor cracks.

Fig. 12 is a photograph taken at Nassau Street, Manhattan, where certain plates were removed from the bottom of the cast-iron lining. It shows very well the gravel packing in the annular space left by the tail of the shield.

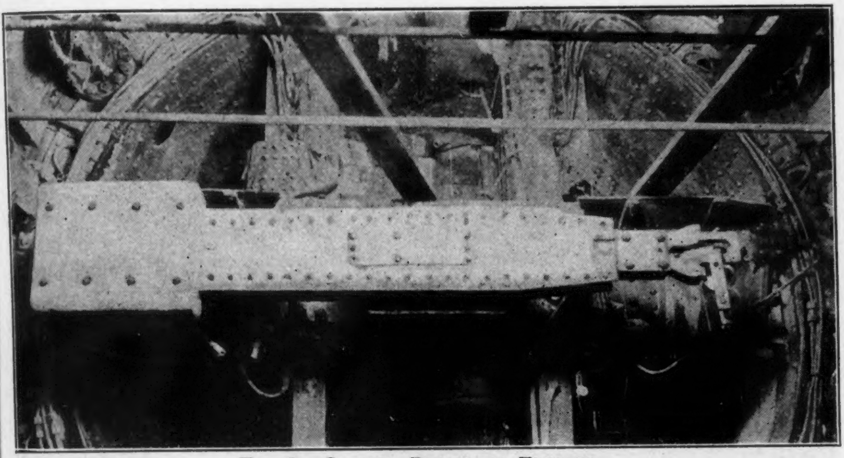

FIG. 9.—OUTSIDE DETAIL OF ERECTOR.

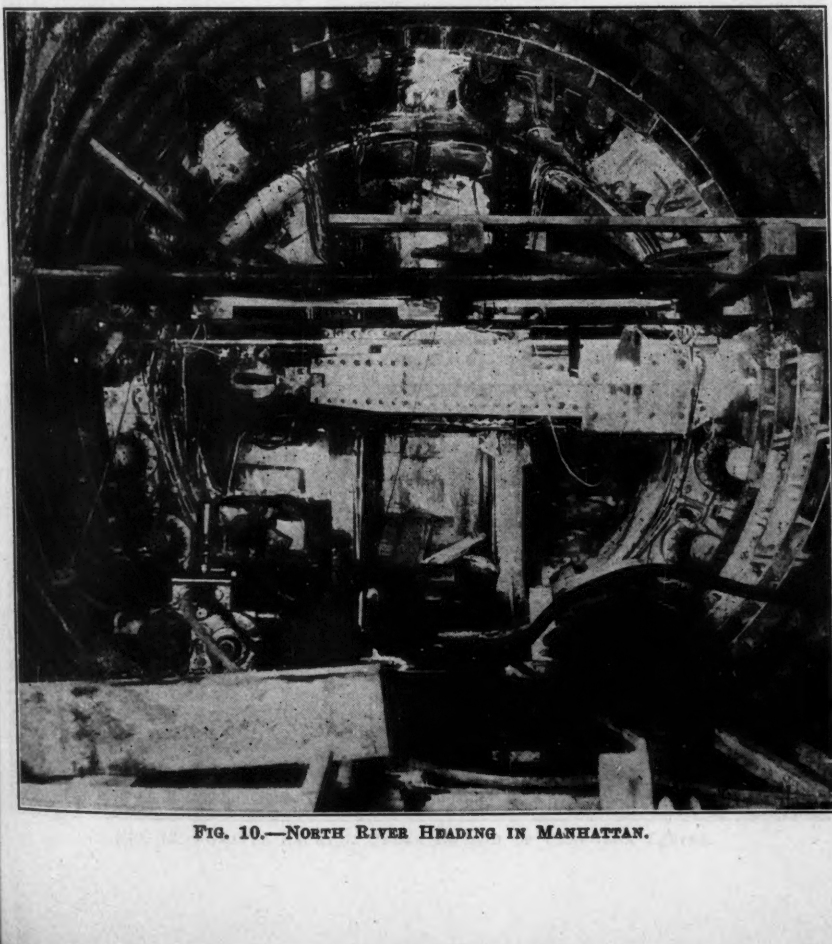

FIG. 10.—NORTH RIVER HEADING IN MANHATTAN.

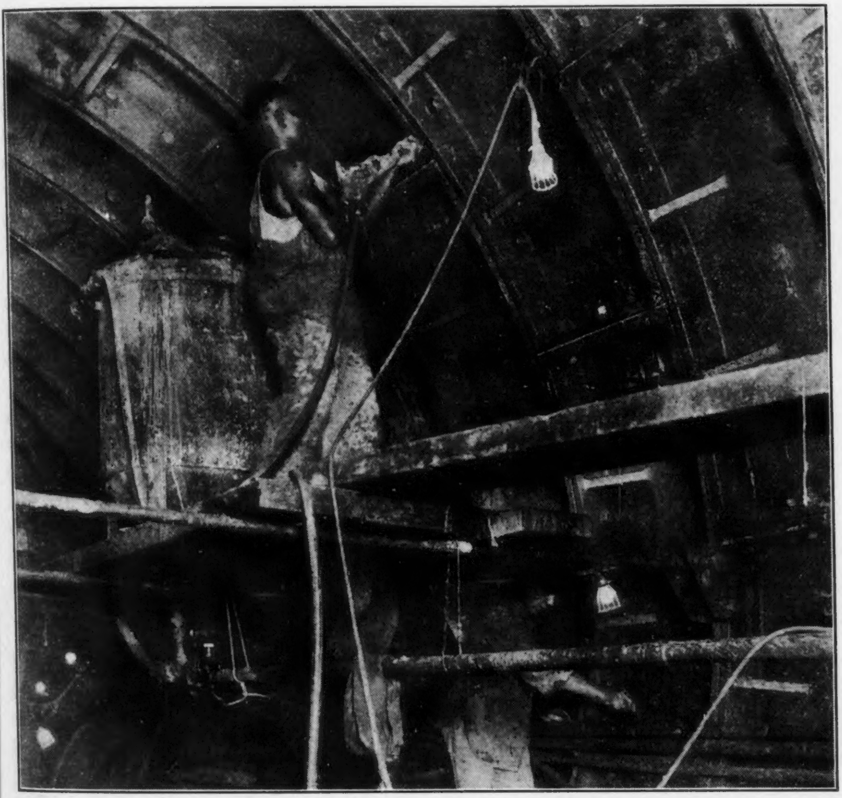

FIG. 11.—VIEW SHOWING GRAVEL PAN.

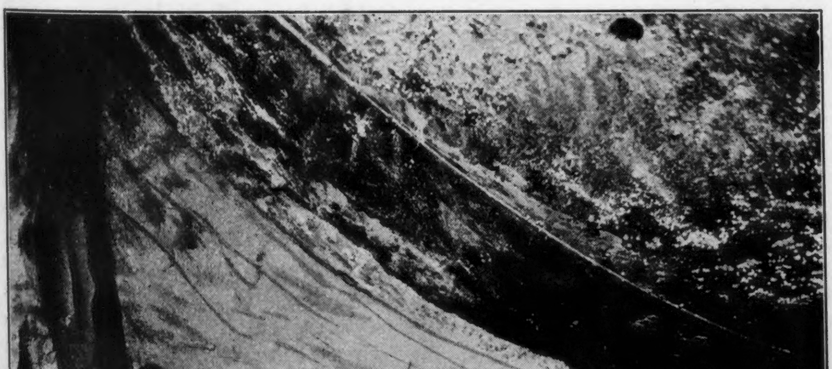

FIG. 12.—GRAVEL PACKING UNDER BOTTOM OF CAST-IRON LINING.

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The ejection of grout outside the cast-iron lining is a second line of defense in the prevention of settlement and is no less important than the gravel process. While graveling is done in the first ring behind the tail, grouting is done seven to twenty rings back. Grout penetrates farther than gravel. It spreads out into seams that may have opened up in the overlying ground. It sometimes spreads out in such a seam over a wide area and, being under pressure, can actually lift up the ground that has slumped down. The grout pan is practically the same as that used in the construction of the Pennsylvania Tunnels.[8]

The machine is nothing but a tank into which sand, water, and cement are dumped. These ingredients are mixed by paddles turned by a small air motor at the end of the machine. The liquid grout is ejected by admission of compressed air after the tank has been closed.

Tunnels are grouted to check the loss of air while the shield is passing through loose ground even when the question of settlement of buildings does not enter. Thus, grouting was extensively used in passing under the dock bulkheads and, also, out under the river. Fig. 12 shows typical shoring beneath an elevated railway structure in Brooklyn, which was required while the tunnel was passing beneath it. The ground settled about 1 in. and the columns were jacked up and held by wedging.

Dock Plant and Gantries

Practically all the materials used for construction were delivered to the job by water transportation and excavated material was disposed of in the same way. The right to use certain property on the East River was given in the contract. In Manhattan, a gantry was built from the shaft, east along Fulton Street to South Street, thence east along the northerly half of Pier 16 (see Fig. 1) to dumping boards erected along the edge of the pier.

The dock space available in Manhattan for handling segments, grouting sand, pea gravel, and cement is shown in Fig. 1. Tunnel segments at one time were stored on the street surface in two tiers for the full width of the space available and back to the edge of the gantry. This iron was kept on hand to guard against the chance of a tie-up of harbor transportation. It would have kept the Manhattan side going for four days if all deliveries had been cut off.

A large storage space was utilized in the yards of the Central Railroad of New Jersey, in Greenville, N. J. When the foundries were producing tunnel segments faster than the contractor could use them, the surplus was placed in storage in New Jersey; when consumption and production were balanced, the segments came directly through to the job, and, finally, when consumption exceeded production, the needed additional rings were ordered out of storage. When the work was going at its maximum pace, the contractor erected about 48 rings per day; this is about 9 carloads.

At the right in Fig. 13 is the cement storehouse, in one corner of which there was a tank machine for soaking grommets in a mixture of red lead, white lead, and linseed oil. The grommets were strung over rods that fitted

[8] *Transactions*, Am. Soc. C. E., Vol. LXVIII (September, 1910), p. 466.

into a frame rotated about a horizontal axis by a small motor. Each batch was turned slowly through the solution of lead and oil for about an hour, when the grommets were removed and hung up to dry, while the solution was replenished and a new batch put into the machine. At the left edge of Fig. 13, one can see the corner of the timber bins in which the pea gravel and grouting sand were stored.

The detail of the measuring hoppers through which the pea gravel and grouting sand were drawn off from the bins and bagged, is shown in Fig. 14. The storage bins were equipped with steam coils for winter conditions. The measuring and bagging device included a screen through which the gravel and sometimes the sand was passed. It was essential to avoid large stones, which choked the hose and caused much delay. The bagged sand and gravel were loaded into muck cars which were elevated to the gantry level in a separate cage constructed near the cement shed. In Brooklyn, the gantry tracks were laid through the third floor of a warehouse. Outside the warehouse the gantry extended out on Pier 6 (see Fig. 1).

Storage-battery locomotives were used throughout the entire job—both on top and in the tunnels. In Manhattan, the batteries were re-charged in the tunnel. A power room was constructed at the top of the shaft with transformers and a motor-generator set. The direct current was carried down the shaft to a re-charging station just inside the locks in each tunnel. Automatic devices prevented the overcharging of any battery. In Brooklyn, the re-charging was handled differently. The batteries were put on flat cars and sent out of the tunnel for re-charging at a station on the gantry.

Shaft Plant

The shaft plant on a tunnel job is required to take care of two very different shaft conditions. There is, first, the period when the shields are being started from the shaft. The deck is in place in its upper position and the space below is under compressed air. All materials pass in and out through locks above the caisson deck. After one tunnel has been extended about 150 ft. from the shaft, a concrete bulkhead is built in the face of the shield, and the air pressure is removed from the heading. The deck is then removed from the caisson, and the second shield is built in the bottom. The deck is replaced and the second shield starts from the shaft in the opposite direction.

After all headings have been started and temporarily bulkheaded, the shaft plant undergoes a complete change. Concrete bulkheads with locks are constructed in all the tunnels. Henceforth, the shaft will be in normal air and the compressed air will be beyond the tunnel bulkheads.

Steel framework is then built up from the bottom of the shaft to above the gantry level. This frame forms a hoistway and supports the cage hoists above the well. On the Fulton Street Tunnels, the cage hoists were operated by push buttons. This was the first time that this type of equipment was used, and it worked out very well. The cages were of the usual design. When it was desired to send a cage down a push button marked "up" raised the cage about 1 ft., freeing the "landing dogs." The "down" button was

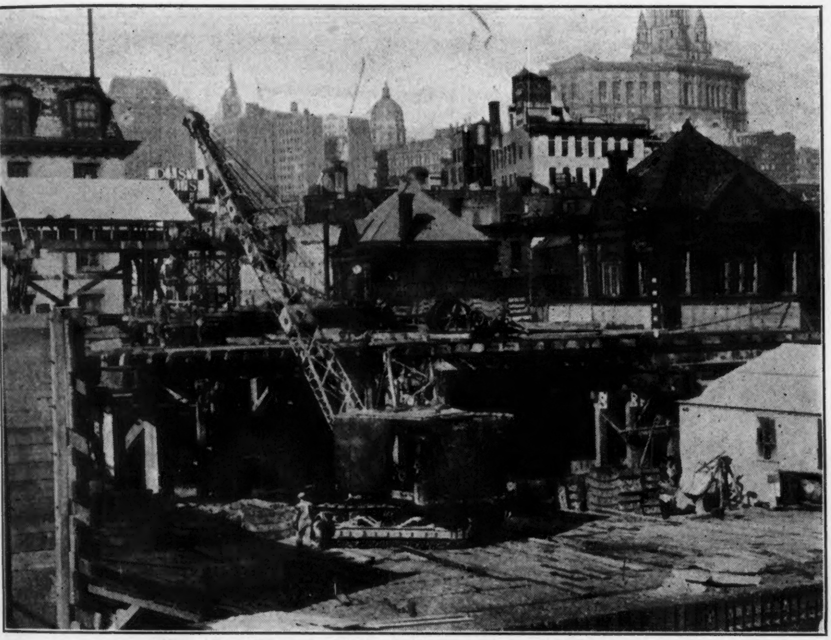

FIG. 13.—STORAGE YARD AT BULKHEAD IN MANHATTAN.

FIG. 14—HOPPERS FOR SCREENING, MEASURING, AND BAGGING GROUTING SAND AND PEA GRAVEL, IN MANHATTAN.

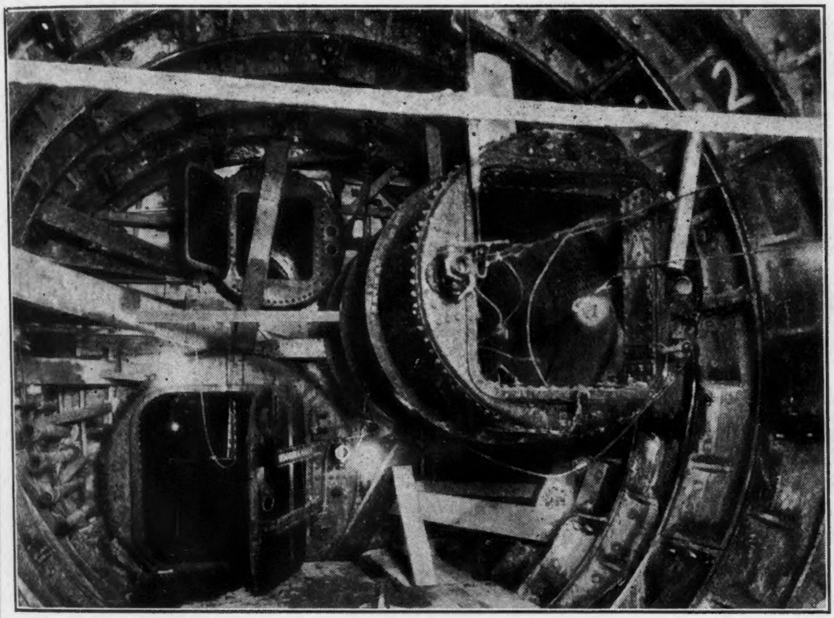

FIG. 15.—BULKHEAD AND LOCKS IN RIVER HEADING.

FIG. 16.—MEETING OF TWO HOODS, MAY, 1929.

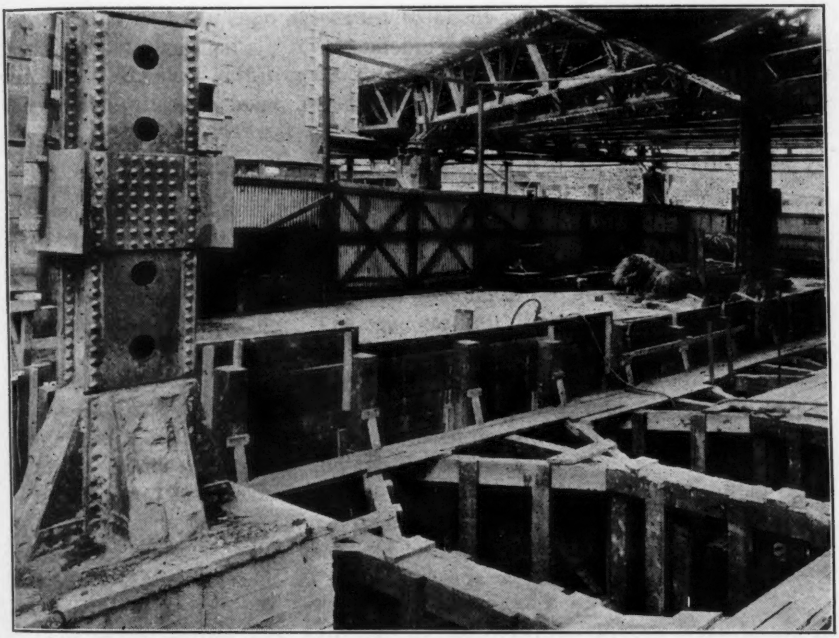

FIG. 17.—EXCAVATION FOR ESCALATOR PITS, HIGH STREET STATION, BROOKLYN.

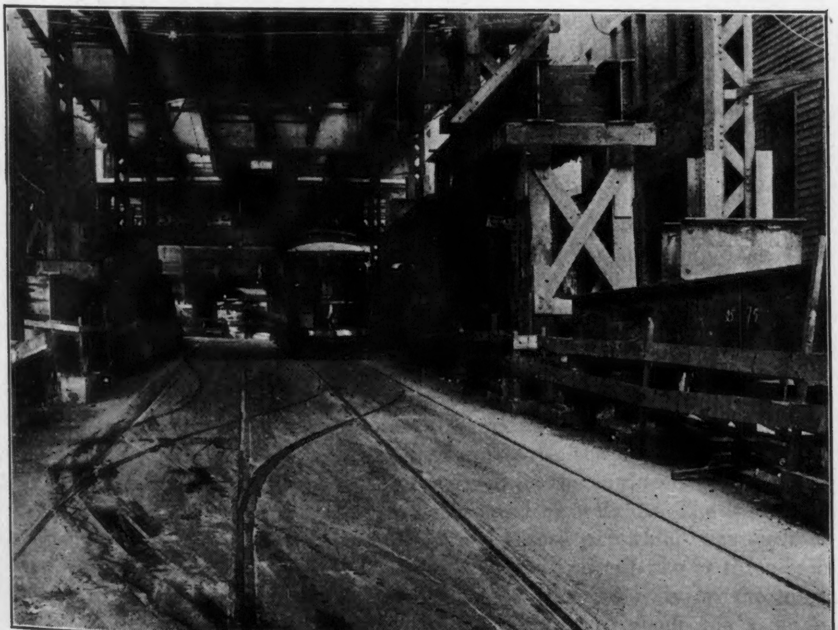

FIG. 18.—SHORING UNDER ELEVATED RAILWAY ON HIGH STREET, BROOKLYN.

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then pushed and the cage descended to about 2 ft. above the bottom where it stopped automatically. A second button was then pushed and the cage was slowly "inched" down to the final level. Each cage had its own 30-h.p., single-drum, electric hoist, which was operated by a slip-ring motor, with solenoid brakes on 220-volt, 3-phase, 60-cycle current at 860 rev. per min.

Tunnel Bulkheads and Safety Screens

The tunnel bulkheads presented no novel features. Because of the large sized cars used, the muck locks had to be 8 ft. 6 in. in diameter. They were 38 ft. long. The man locks were 6 ft. 6 in. in diameter and 34 ft. long. The emergency lock, placed as high as possible in the bulkhead, was 5 ft. 0 in. in diameter and 20 ft. long. The concrete wall of the bulkhead was 6 ft. 6 in. thick in the land headings and 8 ft. thick in the river headings (see Fig. 15).

In addition to its locks, each bulkhead contained two 10-in. air lines; two 6-in. blow lines; one 4-in. high-pressure, air pipe; one 3-in. water line; two $1\frac{1}{2}$-in. hydraulic lines; a back-pressure pipe; and several iron conduits for electric cables and telephone wires.

After the river shields had advanced about 1 000 ft. from the tunnel bulkhead near the bottom of the shaft, a second bulkhead was built close behind the shield. After this bulkhead had been put in use, the air pressure in the section of tunnel between the two bulkheads was maintained at about 15 lb. per sq. in.

The safety screen was required, by specification, in the river headings after the shield had passed the river bulkhead. This is an air-tight bulkhead closing the upper part of the tunnel. Two were used in each river heading so that one would always be in place while the other was being moved ahead. In case of a blow-out and inrush of water, the upper part of the tunnel on the landward side of the screen would be a safe haven for the men if they succeeded in reaching it.

Completion

The south river shields met in May, 1929. Fig. 16 shows the two hoods coming together. After all the excavation was completed and all tunnel lining was erected, there was the big job of cleaning out and caulking. This work was done in normal air. Toward the end of August, the placing of the concrete lining was begun; it was completed in June, 1931.

The mixing plants were carefully planned to secure maximum economy and uniform quality of concrete. A stiff-leg derrick was erected at the bulkhead to unload the sand as it came in on the scows. Such space as was available was utilized for storage piles. A mixing plant was set up near the edge of the gantry. The aggregates were raised by the derrick to a storage bin about the measuring hoppers. As requested by the engineers of the Board of Transportation, the aggregates were measured by weight. The water was measured by volume. The mixer discharged into a car which was elevated to the gantry level and then taken to the form in a train of two or three cars. The concrete cars were made by substituting a concrete body on the muck cars. The mixing plant and methods of placing were about the same in Brooklyn as in Manhattan.

High Street Station

A somewhat unusual feature of the contract was the station to be constructed under High Street, in Brooklyn. The station itself is an island platform about 700 ft. long, with a mezzanine floor above for most of its length. Access to the station is by escalators at Fulton Street and near Adams Street. The escalator pits were excavated from the surface, using horizontal sheeting methods. The elevated railroad columns immediately adjoining the pits were underpinned to sub-grade before the main excavation was made. The column in the foreground of Fig. 17 was first underpinned to sub-grade, but the pier had to be partly removed and the column given a new bearing on top of the completed structure.

Fig. 18 shows the shoring of the steel columns of the elevated railway on High Street preparatory to making the excavation for the Adams Street escalator. After providing for such column shoring, and for trolley, vehicular, and pedestrian traffic, there was not much room left to work. From the bottom of the Fulton Street escalator pit a timbered drift was put through to a point directly above the two cast-iron tubes. From the sides of this drift, mining gangs broke in and started the long station drift. The methods used in this work were new. Since their success depended on the correctness of the designer's figures and not on precedent, there was a certain element of daring in them.

The construction was planned in several successive steps. The first was to pour the tunnel inverts and erect thereon timber bracing for the cast-iron lining. The long station drifts were excavated by miners. Pressed-steel liner plates were used and were supported by arch ribs resting on the tops of the cast-iron tubes and strengthened by a bowstring. The first heading was large enough for the central or bowstring section of the arch rib; side headings followed at a distance of 50 ft. and permitted extending the arch rib down on each side to a firm bearing on the cast-iron tubes. All posting could then be removed and the permanent steel erected. The arch ribs were finally jacked up so that the roof load was taken from the tubes and transmitted down to sub-grade through the permanent steel.

Conclusion

The foregoing covers the major items of plant used on this work. Several new items would be welcome, but have not yet been devised. A number of attempts have been made to develop a bolt-tightening machine but, to date (1930), this problem had not been solved. A small, air-driven tightener that ran up the nut was used, but it did not have the power to complete the job. The final tightening was done by two or three men pulling on a 5-ft. hand wrench with ratchet attachment. Belt loaders were used for a time, but the available space was so small that the advantages of the loader were outweighed by its disadvantages.

Acknowledgments

The construction work described in this paper was done by the Mason and Hanger Company, Incorporated.

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DISCUSSIONS

DESIGN OF A REINFORCED CONCRETE SKEW ARCH

Discussion

BY BERNARD L. WEINER, ASSOC. M. AM. SOC. C. E.

BERNARD L. WEINER,[17] ASSOC. M. AM. SOC. C. E. (by letter).[17a]—In engineering literature too little attention is given to the study of internal stress distribution. The design of any structure may be divided into two parts:

(1) The computation of the reactions, and from the reactions, the determination of the total moments, thrusts, and shears acting at the critical sections of the structure.

(2) The determination of the distribution of the unit stresses produced by the total stresses, in order that the sections may be proportioned to resist the total moments, thrusts, and shears within the allowable unit stresses.

Much has been written on the first step, but rather scant attention has been paid to the second step. Scientific analysis has been replaced repeatedly by more or less arbitrary assumptions, yet the mathematical tools necessary for the proper analysis are in many cases available. Even for the case of the straight beam for which the formulas, $f = \frac{Mc}{I}$, and $v = \frac{VQ}{bI}$, apply, the unnecessary assumption is usually made that a plane section before bending remains a plane after bending. In this case, that assumption happens to be true, but it would be more scientific to prove it rather than merely to assume it. The proof also happens to be simple. What is true of structures, in general, is also true of the skew arch, in particular.

In 1924, J. Charles Rathbun, M. Am. Soc. C. E., published his masterly exposition of the analysis of the skew arch.[18] About two years later he revised and completed his theory as consultant for the Westchester County Park

NOTE.—The paper by Bernard L. Weiner, Assoc. M. Am. Soc. C. E., was published in January, 1931, *Proceedings*. Discussion on this paper has appeared in *Proceedings* as follows: May, 1931, by Messrs. G. D. Houtman, and J. Charles Rathbun; and November, 1931, by A. A. Eremin, Assoc. M. Am. Soc. C. E.

[17] Asst. Engr. and Structural Designer, Westchester County Park Comm., Bronxville, N. Y.

[17a] Received by the Secretary November 17, 1931.

[18] *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), p. 611.

Commission. This theory formed the basis of Part I of the writer's paper; but this work concerned itself almost entirely with the first step; that is, it gave a method, or rather a mathematical theory, for computing the total moments, thrusts, and shears acting on any section of a skew frame. The second step was touched upon both in the report to the Commission and in correspondence with Professor Rathbun. The suggested treatment of the second step was entirely inadequate and scientifically unsound. The result of the writer's study of this part of the subject is given in Part II. Similar studies can profitably be made for many other types of structures, which are still designed more or less arbitrarily.

Such criticism as has been published deals only with Part I. The writer is rather disappointed on this account for he had hoped that criticism of Part II would bring to light valuable information on the theory of internal stresses, especially on the subject of torsion in reinforced concrete.

Mr. Houtman doubts the validity of Professor Rathbun's theory of skew arches when applied to a wide arch. This part of the theory can scarcely be considered open to reasonable doubt as it has been substantiated by tests on models. These tests were made by Professor Rathbun on plaster models,[19] and by George E. Beggs, M. Am. Soc. C. E., on hard rubber models,[20] and they check closely with the results obtained by computation. More recently, Professor Rathbun made similar tests on a plaster model of a multiple-span skew arch.[21] For design purposes, at least, the skew arch theory for obtaining the reactions, seems to be adequate beyond reasonable doubt.

The writer fails to see the logic of Mr. Houtman's method of analyzing a skew arch. No proof whatever is given for the various statements, except some rather vague references to the theory of least work. Referring to Fig. 36, he says, "there is nothing mysterious about the central rectangular arch, *E B F D*." As a matter of fact, there is a great deal of mystery about this arch. For instance, it is not all clear how the vertical shears acting on the vertical surfaces, *D E* and *B F*, are taken into account. Reference to Part II of the writer's paper, will show that there are also a horizontal shear, a thrust, and a bending moment—the values of which per unit length are variable—acting on these surfaces. It is also not clear what the relation is between the analysis of the right arch, *A B′ C′ D*, and that of the skew structure, or why it is on the safe side to assume that the resultant, *R*, is equal to the resultant of this right arch. Furthermore, Fig. 36 may be redrawn so that the central right arch, *E B F D*, disappears entirely, as shown in Fig. 39. Following Mr. Houtman's method, the problem is changed from the design of one skew arch, to that of a smaller skew arch and two triangular arches. Fig. 39 may even be drawn so that the lines, *B F* and *D E*, will coincide. In this case two arches which cannot support themselves will have to support each other. This method of analysis will not bear close scrutiny.

[19] "Crown Stresses in a Skew Arch," *Transactions*, Am. Soc. C. E., Vol. 94 (1930), p. 135.

[20] *Loc. cit.*, Vol. 88 (1925), p. 1208.

[21] "An Analysis of Multiple-Skew Arches on Elastic Piers," Part IV, *Proceedings*, Am. Soc. C. E., April, 1931, Papers and Discussions, p. 539.

It is true that skew arches have been designed and built in entire ignorance of skew-arch principles and that only a few have failed. Of those that have not failed, nothing is known about their factors of safety, nor is anything known about their economy of design. Mr. Houtman stated that "usually failure resulted from excessive compression at the obtuse corner * * *." In view of the skew arch theory, the correctness of this diagnosis is doubtful. The prevailing opinions regarding the intensity of the compression at

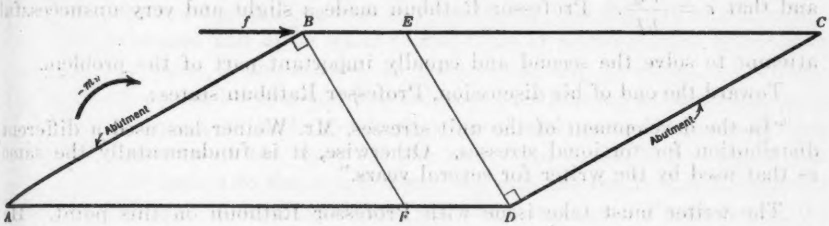

FIG. 39.

the obtuse corner seem to be very much exaggerated. In the writer's notation, the moment which causes this compression is m_u. Referring to Table 8 of the main paper, the dead load value of this moment is − 2 071 kip-ft. and acts as shown in Fig. 39.

The stress, f, treating the arch as an unreinforced beam, is, in pounds per square inch:

$$f = \pm \frac{6\, m_u}{b^2 t} \sec^2 \theta = \pm \frac{6 \times 2\,071\,000 \times 12}{(66.9)^2 \times 144 \times 54} \times 2.42 = \pm 10.35$$

The numerical value of this moment is large, but since the entire width of the arch acts as the depth of a beam to resist it, it follows that the resulting unit stress is small. Furthermore, the peculiar cracks at the acute corners of existing skew arches, to which Mr. Houtman refers, can scarcely be due to tension because of skew arch action, since the longitudinal reinforcement which is designed to resist the ordinary bending moment, M_z, will take up this tension in the action of the arch as a beam the depth of which is the width of the structure. Furthermore, it has just been shown that the estimates of the intensity of this tension is probably much exaggerated.

It is well to point out in this connection, that if the arch is poured in longitudinal strips, care must be taken that each section be well keyed in to the adjacent section by both horizontal and vertical keys, in order to provide adequate shear resistance. Lack of such shear resistance, of course, will destroy the ability of the arch to act as a single beam of depth, b (skew width of the arch).

In the second paragraph of his discussion, Professor Rathbun uses the expressions, "this theory," and "this method of analysis," rather loosely. What Professor Rathbun is referring to is his own theory for obtaining the abutment reactions (and from them the total stresses at any other section), which is given in a revised form in Part I of the writer's paper. The tests mentioned, prove only one thing; that is, that the relation between the loads

and the reactions is correct. The tests prove this, and no more; but as the writer stated in the beginning of this discussion, this is only one of two steps in any complete analysis. It is easy enough to find the reactions in a simple I-beam, and also the maximum moment and shear; but this would be of little value in finding the unit stresses if one did not know that $f = \frac{My}{I}$, and that $v = \frac{VQ}{bI}$. Professor Rathbun made a slight and very unsuccessful attempt to solve the second and equally important part of the problem.

Toward the end of his discussion, Professor Rathbun states:

"In the development of the unit stresses, Mr. Weiner has used a different distribution for torsional stresses. Otherwise, it is fundamentally the same as that used by the writer for several years."

The writer must take issue with Professor Rathbun on this point. He has been familiar with Professor Rathbun's method of stress distribution[22] for some years and disagrees entirely with that method. Obviously, since Professor Rathbun devotes only about four pages to his method, and the writer has thought it necessary to devote about sixty pages to this part of the subject (which forms Part II of his paper), there must be some fundamental difference between the two theories. The results obtained by the use of the two methods are radically different.

In this discussion it is not necessary to go into the differences between the theory of unit stress distribution as given in Part II and Professor Rathbun's method to which reference has been given. To do so would be mere repetition. Some of the more vital points have been discussed by the writer[23] in his criticism of the aforementioned paper by Professor Rathbun. Suffice it to say that the difference in the torsion formula used is only of minor importance. As the writer has remarked several times, the subject of torsion in reinforced concrete is still an open question; and much work remains to be done in this connection. Furthermore, in his development of the theory for proportioning the sections, the writer has given ample proof for each individual step and has not relied on mere assumption. The comparison, therefore, can be easily made by the interested reader.

As pointed out in the reply to Mr. Houtman's discussion, there is a difference of stress between the obtuse corner and the acute corner of the abutment, and the stresses are compression and tension, respectively. The general opinion as to the magnitude of this stress, however, is exaggerated, as shown by the numerical example which gives an indication of the range of the magnitude of these stresses. Professor Rathbun is in error in stating that his equations do not substantiate this belief. This belief is correct as to the presence of the stress, but not as to its magnitude.

In his discussion of Part I, Professor Rathbun is correct in stating that the practice of dropping terms of low values should be treated with

[22] "An Analysis of Multiple-Skew Arches on Elastic Piers," Pt. II, *Proceedings*, Am. Soc. C. E., April, 1931, Papers and Discussions, p. 524.

[23] *Proceedings*, Am. Soc. C. E., September, 1931, Papers and Discussions, p. 1107.

caution. The writer expressed the same idea in his paper, and stated specifically that the question should be looked into carefully by the designer.

Professor Rathbun does not see what is to be gained by the changes in his part of the theory. The writer used Professor Rathbun's unrevised manuscript for a period of about two years. Blue-print copies were furnished by A. G. Hayden, M. Am. Soc. C. E., to many bridge designers all over the United States and abroad. It was found after this period of trial that, for one thing, the system of signs was very confusing and difficult to remember. Constant reference had to be made to the table of nomenclature in order to interpret the signs of the stresses in terms of actual directions. Professor Rathbun referenced his system of co-ordinate axes and his system of signs to directions in space, with the result that forces and moments changed signs with the direction of the skew for the same type of loading. The writer referenced his system to the actual structure, with the result that the stresses for the same type of loading always had the same sign. Furthermore, the sign system was organized in order to make it easier to remember. It was not a question of mathematical correctness, but one of practical convenience.

Professor Rathbun errs in saying that the direction of T_y makes it inconsistent with the idea that the derivative of the moment is the shear, and that it is inconsistent with the definition of shear in simple beams. The common definition of the vertical shear in a simple beam would make T_y act as shown in Figs. 4 and 5. Furthermore, from the equations it follows directly that $\frac{\partial m_z}{\partial v} = t_u$, and this expression is introduced several times in Part II. As a matter of fact, it would make no difference if the direction of the shear, $T_y\,(t_u)$, were reversed. The most serious consequence would be that $\frac{\partial m_z}{\partial v}$ would equal $-t_u$. This might perhaps be inconvenient, but not incorrect. It might also be pointed out that Professor Rathbun, too, is guilty of using double signs for T_y and M_y. This is not caused by the direction of the stresses, but by the simplification made on account of symmetry, and applies only to symmetrical arches. He is in error, however, in including T_z in this category.

Mr. Eremin is correct in adding the terms containing Wc for eccentric loading. The reactions thus obtained for this type of loading would undoubtedly be correct; but whether this would be of any practical value at present is questionable, because the problem of concentrated load distribution would then have to be taken into consideration. This problem is not confined only to the skew arch; it is present also in the right arch. It is a question whether a single line of trucks near the curb of a bridge might not perhaps produce greater stresses at some points than are obtained for the bridge fully loaded.

In his discussion[24] of Professor Rathbun's paper on "Multiple-Skew Arches," the writer has pointed out that the tests made by E. F. Gifford, Assistant Engineer in the Engineering Department of the New York Central Railroad, showed in general that for eccentric loading there are two types of

[24] *Proceedings*, Am. Soc. C. E., September, 1931, Papers and Discussions, p. 1107.

failure: (1) A local failure in the immediate vicinity of the load; and (2), a general failure at points far enough away from the load so as not to be affected by the local load distribution stresses.

For sections where the maximum moments occur with the load at the section, the local load distribution stresses become important since so little is known about them. For this reason knowing the correct reactions for eccentric loading does not help a great deal. Here, again, the all-important question of internal stress distribution enters, and even though the total stresses are available, the unit internal stresses still remain a mystery For the sake of completeness, it is well to include the terms representing the eccentricity of the load and the writer is grateful to Mr. Eremin for pointing this out and for supplying the missing terms.

The application of the skew arch equations to a rigid frame is not a departure from the fundamental requirements of the elastic theory, except that the stresses at the corners are unknown. For the right rigid frame the effect of the corner on the stresses in general has been found to be negligible by comparing the analytical results with those obtained by use of the Beggs' deformeter gauges. Before the first rigid frame was built, the corner was tested for strength, and it was found that failure occurred everywhere, except at the sharp corners where it was predicted.

Mr. Eremin is in error in his interpretation of Fig. 12(*b*) and Fig. 14. These two diagrams, as well as Equations (30), (31), and (32), apply only to the face of the bridge. Nothing is said at this point about the value of the unit shear toward the middle of the bridge. Mr. Eremin is correct in surmising that the shear at the sharp corner of the section is probably not carried uniformly across the section. This is proved by Equations (35) which give the value of the unit horizontal shear at any point of the section, and similar proofs are given for the vertical shear and shear moment per unit width. The angle, θ, does not vary since it is the skew angle. What Mr. Eremin has in mind is the angle, ψ, which gives the direction of the resultant stress at any point. At the face of the bridge where Fig. 12(*b*) and Fig. (14) apply, $\psi = 90° - \theta$, and the resultant stress is parallel to the face of bridge, as it should be.

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DISCUSSIONS

RUN-OFF—RATIONAL RUN-OFF FORMULAS

Discussion

BY ALBERT R. ARLEDGE, ASSOC. M. AM. SOC. C. E.

ALBERT R. ARLEDGE,[31] ASSOC. M. AM. SOC. C. E. (by letter)[31a]—The authors, undoubtedly, have presented a more complete analysis of the subject with which they deal than may be found elsewhere. The paper constitutes a distinct contribution to the profession in each of the two parts into which it is logically divided.

Part I subjects existing methods of estimating the probable maximum run-off of a given area to an orderly and critical analysis, enabling the reader to visualize the many possible variations of the several factors involved. In so doing it brings to light some basic principles which seem to have been entirely neglected by those who have written about the rational method of computing run-off.

Among the hitherto neglected principles may be mentioned: (*a*) Use of the velocities in determining time in the detail method, which do not obtain under the assumptions; and (*b*) the effect on the run-off coefficient of a change in intensity.

The latter principle is by far the more important, especially for the consideration of water-sheds containing considerable previous area. The simple nature of this phenomenon (after its able presentation by the authors), which may so vitally alter the resulting run-off, causes one to wonder why so many able writers on the method have overlooked it. Although it makes the estimation of run-off by the detail process more tedious, its effect, once understood, should certainly be recognized. The "Suggestions (*a*) and (*b*)," and "Comment" forming the conclusion of Part I, may well be heeded by engineers who find it advisable to use the detail process.

NOTE.—The paper by R. L. Gregory and C. E. Arnold, Associate Members, Am. Soc. C. E., was published in April, 1931, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: September, 1931, by Messrs, Le Roy K. Sherman, Francis Bates, and John W. Raymond, Jr., and November, 1931, by Messrs. Reginold A. Ryves, G. S. Tapley, W. I. Hicks, John M. Kemmerer, Carl H. Reeves, Leonard L. Longacre, G. H. Hickox, and Donald M. Baker.

[31] Asst. Civ. Engr., Dept. of Water and Power, City of Los Angeles, Los Angeles, Calif.

[31a] Received by the Secretary November 21, 1931.

Part I, as a whole, should serve the Engineering Profession as a valuable reference for study by those of its members who contemplate the solution of maximum storm flows regardless of the method subsequently adopted.

In Part II of their paper, the authors present run-off formulas which are entirely logical in derivation, as far as the writer is able to determine. It would seem that their presentation at this time, goes far toward satisfying a need of many engineers for a quick and direct method for applying rational principles to the solution of discharges for the larger areas. In this connection attention should be called to (*a*) the paper[32] by Merrill M. Bernard, M. Am. Soc. C. E., together with subsequent discussions thereof; (*b*) treatment[33] of this subject by Harrison P. Eddy and the late Leonard Metcalf, Members, Am. Soc. C. E.; (*c*) discussion[34] by Charles H. Lee, M. Am. Soc. C. E., of the paper by Oren Reed, Assoc. M. Am. Soc. C. E.; and (*d*) writings by others in the general literature on the method.

The saving of time through use of the formulas, as compared with that of the detail process, will naturally depend not only on the nature and extent of the areas, but also on the purpose of the engineer. Generally, the time saved will vary with the size of the area above the point at which discharge is sought. It is unfair, therefore, to compare these formulas (without any qualifications) solely with the detail method of estimating storm flow for a small city drainage system, wherein the discharge is sought at every inlet point. Even for small areas (and likewise for any irregular area) the formulas should prove time savers in making trials for the determination of run-off from water-sheds, a part of which will render a greater value under the assumptions, than the whole.

Equation (39) deserves special mention. This formula appears to embody the culminating thoughts of the authors, in their analysis of the run-off coefficient, especially as regards future experimentation. Indeed, it would seem that the "rational method" should scarcely be called rational without possession of Equation (39), or its equivalent; and it should prove valuable in future experimentation which contemplates the use of any method for calculating discharges.

In conclusion, the writer wishes to state that the authors deserve the appreciation of the profession, not only for the great energy and thought necessarily required for the creation of their paper, but also for the actual value of the gift itself. The analysis, conclusions, and formulas therein, will be referred to in the future. The future worth of the formulas will depend largely on the amount of intelligent use to which they are subjected. The authors, no doubt, will be able to make further suggestions; and encouragement to them, as well as to others who are able and willing to use their time for constructive thought, should be freely rendered by the profession.

[32] "Formulas for Rainfall Intensities of Long Duration," *Proceedings*, Am. Soc. C. E., October, 1930, p. 1835.

[33] "American Sewerage Practice," Vol. 1, Second Edition, pp. 305–306.

[34] *Transactions*, Am. Soc. C. E., Vol. 93 (1929), p. 433.

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DISCUSSIONS

MANUFACTURING CONCRETE OF UNIFORM QUALITY

Discussion

BY MESSRS. O. BONNEY AND D. T. MITCHELL, C. E. ARNOLD, AND MORRIS MASON

O. BONNEY,[16] M. AM. SOC. C. E., AND D. T. MITCHELL,[17] JUN. M. AM. SOC. C. E. (by letter).[17a]—This paper presents and discusses a subject which is of interest and importance to all who have to do with the design and construction of concrete structures. As indicated by the author concrete of uniform quality has not always been produced in the past and experiences and information which may lead to more uniform concrete in the future are welcomed by engineers. When the art of making concrete reaches the state where uniformity is the regular occurrence it may then perhaps be possible to design structures of this material more economically.

The author emphasizes the fact that the production of concrete of uniform quality requires a thorough knowledge, on the part of all concerned, of the ingredients as well as a thorough knowledge of proportioning, mixing, transporting, placing, and curing. The engineer and the contractor are vitally concerned and 100% co-operation is required of both.

The author sets forth under the caption, "Irregularities in the Strength of Concrete," the principal causes of the great variations that occur in "job" concrete. In this connection the writers feel that the method of mixing is a matter of considerable importance and, furthermore, that careful consideration should be given to the type of mixer used, the peripheral speed of its drum, the design of the mixing blades, the size of the drum, the mixing action, the time of mixing, and the ratio of the quantity of concrete mixed per batch to the volume of the drum.

Until recent years it has been a common practice to mix concrete in non-tilting, rotary, drum type mixers. Since about 1928, however, a type known

NOTE.—The paper by William M. Hall, M. Am. Soc. C. E., was published in May, 1931, *Proceedings*. Discussion of the paper has appeared in *Proceedings*, as follows: September, 1931, by Messrs. Henry B. Seaman, Theodore Belzner, John Sanford Peck, and J. W. Kelly; and November, 1931, by Messrs. Stanton Walker and Edward E. Bauer.

[16] Sewerage Relief Engr., Div. of Eng., Columbus, Ohio.

[17] Field Engr., Sewerage Relief, Div. of Eng., Columbus, Ohio.

[17a] Received by the Secretary, October 27, 1931.

as a truck mixer has been introduced and used on work of large magnitude, which permits the concrete to be mixed while in transit to the point of deposit, if that is desired.

Truck mixers which have come to the notice of the writers are designed to mix more concrete per batch in proportion to the total volume of the drum than the older types of mixers. Furthermore, the peripheral speed is less in the case of the truck mixer. Definite conclusions as to the effect of this on the quality of the concrete mixed in truck mixers have not been reached by the writers, but tests made on one type indicate that the mixing time for a truck mixer should be greater than that for the older type. Other tests indicated that there may be some variation in compressive strength between concrete taken from the first of the discharge of a batch and that taken from the last of the discharge. Other types of truck mixers might give different results.

During the three years since 1927 a part of what is known as the Olentangy-Scioto Intercepting Sewer has been constructed in Columbus, Ohio; this part is a rectangular, reinforced concrete structure, involving about 34 000 cu. yd. of concrete. Throughout the work attention has been directed toward making concrete of uniform quality, and the writers, therefore, submit the results of their compression tests of concrete as being of interest in connection with the results shown by the author. The results obtained on this work, together with weighted averages of those given by the author, are shown in Table 4.

The work was divided into four sections, each section being a separate contract of the size and length shown in Table 4, Items 21, 22, and 23. With the exception of Section 4 and part of Section 1, the work was constructed in open trenches. Section 4 was constructed along the east bank of the Scioto River, the river side of the sewer being exposed. Approximately, 420 lin. ft. of Section 1 was constructed as a dam across the river.

Three brands of cement were used, one brand on Sections 1 and 4, another on Section 2, and still another on Section 3. The specifications for aggregate were practically the same on all four sections. The fineness modulus of the fine aggregate was required to be between 2.75 and 3.25 and that of the coarse aggregate between 6.90 and 7.50. The fineness modulus of the fine aggregate was usually found to be near the upper limit and, in fact, on Sections 1 and 2 it was necessary to add fine sand in order to obtain a fineness modulus within the limits specified. Ordinarily, the coarse aggregate was found to be satisfactory with respect to the fineness modulus. City water, softened and filtered, was used.

All concrete was mixed for 3 min. in 1-cu. yd. paving mixers of the non-tilting, rotary, drum type. The water content was controlled for the most part by the slump test. The aggregates were measured by volume at the screening plant located from 1 to 3 miles from the mixer, and then hauled to the mixer in trucks which were provided with batch compartments. An inspector was stationed at the screening plant, another at the concrete mixer, and still another at the point of final deposit.

The test cylinders were made in accordance with the Standard Methods of Making and Storing Specimens of Concrete in the Field (Serial Designation: C 31-27), of the American Society for Testing Materials, with the exception that the concrete was obtained as near the point of final deposit

TABLE 4.—COMPARISON OF CYLINDER TESTS OF CONCRETE IN THE OLENTANGY-SCIOTO INTERCEPTING SEWER, AT COLUMBUS, OHIO, WITH THOSE IN DAMS ON THE OHIO RIVER IMPROVEMENT. (28-DAY CYLINDERS ONLY.)

Item (1)	Description (2)	Section 1 (3)	Section 2 (4)	Section 3 (5)	Section 4 (6)	Weighted average (7)	Weighted average of cylinder tests on the Ohio River Improvement (8)
	Results of Tests:						
1	Number of cylinders.........	217	108	407	319	1 051*	759*
2	Average strength per cylinder, in pounds per square inch...	3 460	3 165	3 322	3 444	3 371	2 815
3	Average strength of all cylinders testing less than Item 2, in pounds per square inch..	3 063	2 760	2 777	3 002	2 901	2 315
4	Average strength of all cylinders testing more than Item 2, in pounds per square inch.	3 909	3 599	3 931	3 883	3 877	3 381
5	Percentage of cylinders testing less than Item 2..........	53	52	53	50	52	54
6	Percentage of cylinders testing less than 2 000 lb. per sq. in.	1	0	3	1	2	17
7	Percentage of cylinders testing less than 2 800 lb. per sq. in.	....	....	....	....	....	10†
	Compressive strength, in pounds per square inch:						
8	Maximum..................	4 990	4 210	5 940	5 060	5 940‡	5 930‡
9	Minimum..................	1 590	2 100	1 270	1 730	1 270§	919§
	Slump, in inches:						
10	Average..................	3.5	3.9	4.5	4.8	4.3	3.52
11	Maximum..................	7.0	7.5	8.75	7.75	8.75‡	8.5‡
12	Minimum..................	1.0	0.5	0.5	2.5	0.5§	0.5§
13	Number of tests in which the slump was less than in Item 10..........	108	52	199	159	518*	421*
14	Year constructed..........	1930	1930	1928–1930	1929–1930	1928–1930	1925–1929
15	Volume of concrete placed, in cubic yards..............	12 193	4 115	11 165	6 654	34 127*	750 000*
16	Mix, by volume............	1:6	1:6	1:6	1:6	1:6	(See text)
17	Water, in gallons per sack of cement..................	6	6	6.75	6	....	(See text)
18	Specified minimum strength, in pounds per square inch......	2 500	2 500	2 500	2 500	2 500	2 000‖
19	Percentage of cylinders testing as much or more than Item 18..........	96	91	89	94	92	83
20	Percentage of cylinders testing less than Item 18..........	4	9	11	6	8	17
	Dimensions of Sewer:						
21	Width, in feet............	10.5	10.5	10.5	10.5	....	....
22	Height, in feet...........	5.5	12.0	12.0	12.0	....	....
23	Length, in linear feet......	4 550	1 206	3 718	1 853	....	....

* Total. † Six-month cylinders. ‡ Maximum obtained. § Minimum obtained. ‖ Job standard.

as possible and the cylinders were stored in damp sand on the work for only 24 to 48 hours. At the end of this time they were removed to the testing laboratory where they were then stored in damp sand or in a damp room for the remainder of the time until broken. The cylinders were broken in accordance with the A. S. T. M. Standard Methods of Making Compression Tests of Concrete (Serial Designation: C 39-27).

C. E. ARNOLD,[18] ASSOC. M. AM. SOC. C. E. (by letter).[18a]—The author presents some interesting information which indicates that with ample, proficient, and well organized supervision, concrete of quite uniform quality can be manufactured even on detached or widely separated pieces of work.

During 1928 and 1929 the writer installed 41.33 miles of storm drains to intercept and remove the storm waters from an area of 22 319 acres lying adjacent to, and immediately east of, Los Angeles, Calif. The entire system is divided into four main outlet lines, lying several miles apart, the lower (larger) portions of which are of monolithic construction, rectangular in cross-section, ranging from 6 to 10 ft. in height and from 8 to 14 ft. in width; these portions have a combined length of 13.09 miles.

The structures were designed on the basis of 650 lb. per sq. in. for concrete in compression and 16 000 lb. per sq. in. for steel in tension. Naturally the cross-section area of concrete and the weight of steel (per foot of length) varied due to the superimposed load, which was from a few feet to as much as 40 ft. of cover (in which case an arch section was used).

Three of the main lines were built simultaneously, the fourth partly overlapping the construction period of the first three. Combined, they contain more than 141 000 cu. yd. of concrete, an average of a little more than 2 cu. yd. per ft. of length. In all cases the concrete was mixed on the ditch bank in large mixers and conveyed to the structure by a traveling bucket, belt conveyor, or chute. The fine and coarse aggregates were delivered to the mixers in 5-yd. batch trucks, the contents of each batch being weighed as loaded.

In all, 510 pairs of standard 6 by 12-in. test cylinders were made, one-half of which were tested at 7 days and the remainder at 28 days. The strength developed by the 28-day cylinders led the writer to analyze the tests which resulted in the data plotted on Fig. 4. Fig. 4(*a*) is a curve of 510 tests made on 28-day cylinders. The cylinders were made of Cements *A* and *B* and Coarse Aggregates *A, B, C,* and *E;* Fig. 4(*b*) is an analysis of 495 tests segregated to show any difference in strength, due to the brand of cement used; Fig. 4(*c*) is an analysis of 507 tests segregated to show any difference in strength due to varying the brand of coarse aggregate; and Fig. 4(*d*) is the result of plotting curves of the 510 tests in classes according to the slump. It was thought that a higher unit concrete stress (about 800 lb. per sq. in.), might be used in future designs, which seems justified since only 4% of the tests showed strengths less than 2 500 lb. per sq. in. The writer would appreciate the author's comments relative to such an increase.

Inspection.—One chief storm-drain inspector, a graduate from a reputable engineering school, was employed to supervise and to assist, when necessary, the inspectors stationed on the job. The inspectors on the monolithic work were experienced men (some had been with the writer for more than three years), who knew the importance of the water-cement ratio, the fineness modulus, and the necessity for a workable mix. They were extremely loyal and more than faithful.

[18] Storm Drain and Constr. Eng., County of Los Angeles, Los Angeles, Calif.

[18a] Received by the Secretary November 2, 1931.

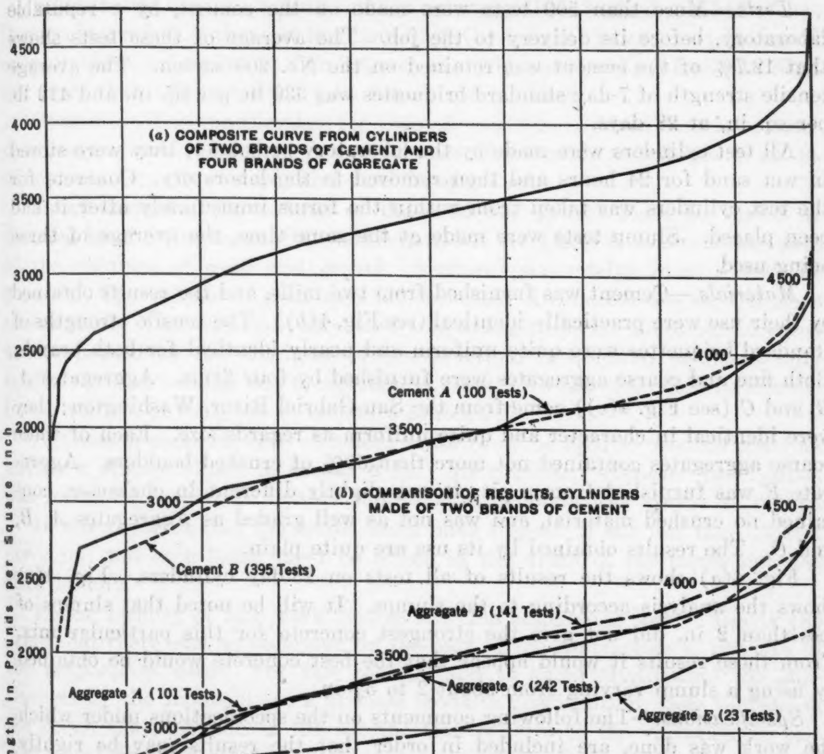

FIG. 4.—COMPARISON OF TESTS RESULTS ON CONCRETE CYLINDERS BY SEGREGATION OF VARIABLES.

Tests.—More than 500 tests were made on the cement, by a reputable laboratory, before its delivery to the job. The average of these tests shows that 12.7% of the cement was retained on the No. 200 screen. The average tensile strength of 7-day standard briquettes was 330 lb. per sq. in. and 412 lb. per sq. in, at 28 days.

All test cylinders were made by the laboratory personnel; they were stored in wet sand for 24 hours and then removed to the laboratory. Concrete for the test cylinders was taken from within the forms immediately after it had been placed. Slump tests were made at the same time, the average of three being used.

Materials.—Cement was furnished from two mills, and the results obtained by their use were practically identical (see Fig. 4(*b*)). The tensile strengths of standard briquettes were quite uniform and nearly identical for both brands. Both fine and coarse aggregates were furnished by four firms. Aggregates *A*, *B*, and *C* (see Fig. 4(*c*)) came from the San Gabriel River, Washington; they were identical in character and quite uniform as regards size. Each of these coarse aggregates contained not more than 30% of crushed boulders. Aggregate *E* was furnished from a pit; it was slightly different in character, contained no crushed material, and was not as well graded as Aggregates *A*, *B*, and *C*. The results obtained by its use are quite plain.

Fig. 4(*a*) shows the results of all tests on 28-day cylinders. Fig. 4(*c*) shows the analysis according to the slumps. It will be noted that slumps of less than 2 in. did not give the strongest concrete for this particular mix. From these results it would appear that the best concrete would be obtained by using a slump varying from about 2 to 3¼ in.

Specifications.—The following comments on the specifications under which the work was done, are included in order that the results may be rightly judged. All cement was required to meet A. S. T. M. Standards (Serial Designation, C9-26). Class A concrete, as defined by the specifications, was used in all reinforced work.

Fine Aggregate.—It was required that fine aggregate for use in concrete or mortar consist of hard, durable, mechanically washed particles resulting from the natural disintegration of rock, having clean grains uncoated with clay, alkali, or oil, and free from organic matter.

When ready for use it was required to be so graded that not more than 6% (by weight), including the loss by decantation (A. S. T. M. Standard; Serial Designation, D136-22T) passed a No. 100 sieve; not less than 17%, and not more than 42% passed a No. 30 sieve; not less than 60%, and not more than 85%, passed a No. 10 sieve; and not less than 93% passed a ¼-in. round opening screen. All passed a ½-in. round opening screen.

Coarse Aggregate.—Coarse aggregate for use in concrete was specified to consist of crushed stone or gravel or a combination of gravel and crushed stone. It must be hard, mechanically washed, and have clean surfaces uncoated with silt, clay, alkali, oil, or organic matter. It must have a specific gravity of not less than 2.50 and must have a French coefficient of not less than 9 when subjected to the Deval abrasion test (A. S. T. M. Standard; Serial Designation, D2-08).

When ready for use, it must contain not more than 30% of crushed stone; it must be graded so that not less than 95% (by weight) passed a $1\frac{1}{2}$-in. round opening sieve, but not more than 7% passed a $\frac{1}{4}$-in. sieve, and not less than 30%, nor more than 55%, passed a $\frac{3}{4}$-in. round opening sieve.

Concrete.—The specification for Class A concrete required that it be composed approximately of the relative volumes of materials as follows: 1 cu. ft. of cement; 2 cu. ft. of sand; and 4 cu. ft. of gravel.

In all concrete, the volume of the sand and gravel could be varied as directed by the Engineer of Construction, but for Class A concrete, the sum of the volumes of sand and gravel (measured separately) must always be 6 cu. ft. The volume of sand specified for Class A concrete was on the basis of dry sand. The volume of any sand containing moisture could be increased as determined by the Engineer of Construction, in quantity sufficient only to allow for the bulking due to moisture content. No increase could be allowed in the volume of rock due to moisture content.

In Class A concrete, only sufficient water could be used to produce concrete having a slump of not to exceed 4 in. All concrete, whenever practicable, was mixed in batch-type mixers and Class A concrete was always so mixed.

It was required that the mixer drum have a speed of not less than 13 rev. per min., and each batch must be mixed in the drum of the mixer not less than $1\frac{1}{2}$ min.

In the locality of this work, coarse and fine aggregates cost so nearly the same that there was no objection on the part of the contractors to changes in the proportions used. For this reason (and to avoid frightening the contractors by the inclusion of a fineness modulus specification, thus increasing the cost), the provision in the specification relative to the variation in percentage of fine and coarse aggregate was included and was freely used by the writer to secure a uniform workable mix. The proportions actually used on the work varied between the following limits: Cement, 1 part; fine aggregate, 2.25 to 2.35 parts; and coarse aggregate, 3.75 to 3.65 parts.

The writer feels that an analysis of the author's tests in a manner similar to that shown on Fig. 4 would be of value to the Engineering Profession. With the present knowledge of the manufacture of concrete and the methods and equipment available for testing the materials, a general increase in the unit working stress is in order. The presentation of such papers as that of Mr. Hall will enable engineers to judge, more correctly, the extent to which an increase seems advisable.

MORRIS MASON,[19] JUN. AM. SOC. C. E. (by letter).[19a]—It has been stated by Mr. Hall that the more scientific methods of concrete control have produced a more uniform concrete. Others have found that a designed strength of concrete may be produced at a lower cost by the application of these methods. If this is true, it is difficult to understand why the application has not become

[19] Engr., Standard Oil Co. (Indiana), Wood River Refinery, Wood River, Ill.

[19a] Received by the Secretary November 18, 1931.

more general; but engineers show a great deal of stubbornness in maintaining formulas and rules which they have found to be usable, although not perfect.

In Appendix II, the statement is made that "hostility is one of the greatest obstacles to the control and inspection of concrete construction." This is the prevalent attitude on many concrete jobs. The engineer often encounters the following attitude among the men in charge of concrete work: "I was pouring concrete twenty years before your method was devised. I have poured good concrete and can continue to do so." Steel was made successfully for many years, by the same type of methods which are now in use in mixing concrete, but no steel company would consider using such methods under present conditions.

A number of methods of concrete control have been used successfully in the past. The Illinois State Highway Department has followed the "mortar-voids" method. The "surface-area" method has been used to some extent and the "water-cement-ratio" method has also been very generally adopted with success. A conclusive proof of the advisability of these methods is given in the fact that when any of the newer methods are used, the old "trial-batch" or "fixed-ratio" methods are not reinstated. It is left to the decision of the engineer as to which of the methods are more usable on the particular work with which he is connected. Even if the application of the method be imperfect, "a poor estimate is better than a good guess." The reports on these methods are unanimous in that more uniform concrete can be produced at a lower cost.

When the practical concrete man has grasped the advantages of these methods he is generally in favor of their continued use, because he sees the improved quality and uniformity of the concrete. Excessively sloppy mixes are slowly being condemned. The presentation of conclusive proof that better concrete can be produced, even under adverse circumstances, as has been shown in this paper, will hasten the discarding of many of the older methods of concrete control.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852

DISCUSSIONS

DESIGN OF LARGE PIPE LINES

Discussion

BY MESSRS. JOHANNES SKYTTE, DONALD E. LARSON, RAYMOND J. ROARK, AND F. W. HANNA

JOHANNES SKYTTE,[18] ASSOC. M. AM. SOC. C. E. (by letter).[18a]—The author is to be commended for producing this paper. It is the writer's belief that this method can be applied with advantage in many cases to the design of pipe lines and other cylindrical structures.

Primary Stresses in the Pipe Shell.—The stress condition will be general only when it involves the radial shears, S_r and S_l, and the moments, M_r and M_l (see Fig. 11). However, S_l and M_l are very small quantities for any partial filling of pipe and will be herein considered equal to zero.

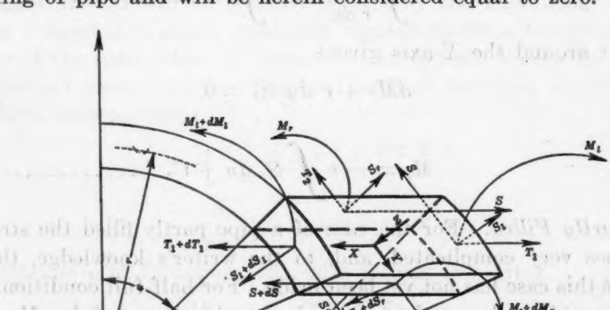

FIG. 11.

For the three cases treated in the paper, namely, (1) dead weight of pipe; (2) pipe just full; and (3) pipe under pressure,

$$S = -C \sin u \qquad (58)$$

in which, C is a constant. Consequently, T_1 has a straight-line variation as for ordinary beams, and S_r and M_r become equal to zero.

NOTE.—The paper by Herman Schorer, Assoc. M. Am. Soc. C. E., was published in September, 1931, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: November, 1931, by Messrs. L. J. Mensch and W. P. Roop.

[18] Asst. Hydr. Engr., Hetch Hetchy Project, San Francisco, Calif.

[18a] Received by the Secretary October 19, 1931.

The general formulas, Equations (59), (60), and (61), are derived by projection on the three axes. Equation (62) is derived by taking the moment around the X-axis.

Projection on the Z-axis gives:

$$Z\,r\,du\,dx + (T_2 + dT_2)\,dx\,\tfrac{1}{2}\,du + T_2\,dx\,\tfrac{1}{2}\,du + dS_r\,dx = 0$$

or,

$$T_2 = -r\left(Z + \frac{dS_r}{r\,du}\right) \quad \text{(59)}$$

Projection on the Y-axis gives:

$$Y\,r\,du\,dx + (T_2 + dT_2)\,dx - T_2\,dx - 2S_r\frac{du}{2}dx + (S + dS)\,r\,du - S\,r\,du = 0$$

or,

$$dS = -\frac{d\,T_2\,dx}{r\,du} + \frac{S_r}{r}\,dx - Y\,dx$$

and, consequently,

$$S = -\int\frac{d\,T_2}{r\,du}\,dx + \int\frac{S_r}{r}\,dx - \int Y\,dx + C_1 \quad \text{(60)}$$

Projection on the X-axis gives, in a similar way:

$$T_1 = -\int\frac{dS}{r\,du}\,dx - \int X\,dx + C_2 \quad \text{(61)}$$

Moment around the X-axis gives:

$$dM_r + r\,du\,S_r = 0$$

or,

$$M_r = -r\int S_r\,du + C_3 \quad \text{(62)}$$

Pipe Partly Filled.—For the case of a pipe partly filled the stress condition becomes very complicated, and, to the writer's knowledge, the general solution for this case has not yet been made. For half-full condition, however, a very thorough and complete analysis has been made[19] by Mr. A. Frey Samsioe, of Stockholm, Sweden, in a volume of about 100 pages, to which reference has also been made by the author.

The same problem was treated by Mr. K. I. Karlsson, also of Stockholm, and although his analysis, as stated by Mr. Schorer, was based on assumptions which may appear rather arbitrary, it gives a very good indication of the nature of the stress condition for this grade of filling, and the following development is based on Karlsson's assumptions.[20]

[19] "Die Spannungen in einem auf mehreren Stützen in gleicher gegenseitiger Entfernung aufgelegten and zur hälfte mit Wasser gefüllten Rohr," by A. Frey Samsioe, Ingeniörs Vetenskaps Akademiens Handlingar Nr. 50, Stockholm, 1926; pub. by Svenska Bokhandelscentralen, Stockholm, 1926.

[20] "Ueber Schwerkraftspannungen in Rohrleitungen von grossen Durchmessern und deren rationelle Kronstruktion," by Karl I. Karlsson, *Schweizerische Bauzeitung*, Vol. 80, September 2, 1922, p. 105.

Consider a slice of pipe of unit length. The sum of the loadings in Fig. 12(*a*) and Fig. 12(*b*) are equivalent to the loadings for half-full condition, or as shown in Fig. 12(*c*), and, consequently, can be substituted for the latter. For the loading shown in Fig. 12(*a*),

$$S_1 = -\tfrac{1}{2}\, q\, r\, x \sin u$$

and the ring moment, M_{r_1}, and, consequently, the deflections, are zero at all points (rib-shortening is not considered). Therefore, in order to find the resulting ring moment, M_{r_1}, one needs only to consider the loading as in Fig. 12(*b*). In this case the deflection will cause a shortening of the horizontal diameter and a lengthening of the vertical diameter.

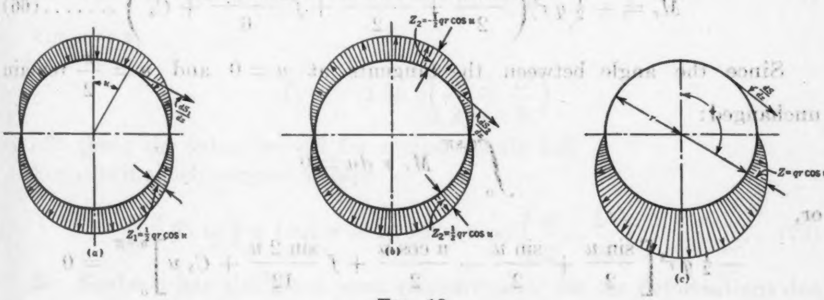

FIG. 12.

This deformation will be resisted greatly by the rigid ring support and will cause a tangential shear, gradually decreasing from the ring support to the center of the span, where it becomes equal to zero. Since the shear must be symmetrical about the two axes and zero at the ends of the axes it is reasonable to assume that:

$$S_2 = f \times \tfrac{1}{2}\, q\, r\, x \sin 2\,u$$

or,

$$\frac{dS_2}{dx} = f \times \tfrac{1}{2}\, q\, r \sin 2\,u$$

Differentiation of Equation (60), with respect to x gives,

$$\frac{dS_2}{dx} = -\frac{dT_2}{r\,du} + \frac{S_r}{r} - Y$$

since $Y = 0$,

$$\frac{dS_2}{dx} = -\frac{dT_2}{r\,du} + \frac{S_r}{r} = f \times \tfrac{1}{2}\, q\, r \sin 2u \quad \ldots\ldots\ldots\ldots\ldots (63)$$

Next, substitute $Z_2 = -\frac{1}{2}\, q\, r \cos u$, in Equation (59), and differentiate with respect to u, thus:

$$\frac{dT_2}{du} = -r\left(\tfrac{1}{2}\, q\, r \sin u + \frac{d^2S_r}{r\,du^2}\right) \quad \ldots\ldots\ldots\ldots\ldots (64)$$

Eliminating T_2 between Equations (63) and (64):

$$\frac{d^2S_r}{du^2} + S_r = \tfrac{1}{2}\, q\, r^2\, (f \sin 2\,u - \sin u)$$

The complete integral is:

$$S_r = \tfrac{1}{2}\, q\, r^2 \left[\frac{u \cos u}{2} - f\, \frac{\sin 2\, u}{3} \right] + C_4\, e^{+u\sqrt{-1}} + C_5\, e^{-u\sqrt{-1}}$$

For $u = 0$ and $u = \dfrac{\pi}{2}$, $S_r = 0$, or, $C_4 = C_5 = 0$, and, consequently,

$$S_r = \tfrac{1}{2}\, q\, r^2 \left[\frac{u \cos u}{2} - f\, \frac{\sin 2\, u}{3} \right] \ldots\ldots\ldots\ldots\ldots\ldots\ldots (65)$$

Substituting for S_r in Equation (62) and integrating:

$$M_r = -\tfrac{1}{2}\, q\, r^3 \left(\frac{\cos u}{2} + \frac{u \sin u}{2} + f\, \frac{\cos 2\, u}{6} + C_3 \right) \ldots\ldots (66)$$

Since the angle between the tangents at $u = 0$ and $u = \dfrac{\pi}{2}$ remain unchanged:

$$\int_0^{0.5\pi} M_r\, r\, du = 0$$

or,

$$-\tfrac{1}{2}\, q\, r^3 \left[\frac{\sin u}{2} + \frac{\sin u}{2} - \frac{u \cos u}{2} + f\, \frac{\sin 2\, u}{12} + C_3\, u \right]_0^{0.5\pi} = 0$$

or, $C_3 = \tfrac{1}{2}\, q\, r^3\, \dfrac{2}{\pi}$, and, consequently Equation (66) becomes:

$$M_r = -\tfrac{1}{2}\, q\, r^3 \left(\frac{\cos u}{2} + \frac{u \sin u}{2} + f\, \frac{\cos 2\, u}{6} - \frac{2}{\pi} \right) \ldots\ldots (67)$$

For a minimum of internal work, f is found equal to 0.85. Substituting this value for f:

$$M_{r0} = -\,0.0050 \times \tfrac{1}{2}\, q\, r^3$$

and,

$$M_{0.5\pi r} = -\,0.0072 \times \tfrac{1}{2}\, q\, r^3$$

For a pipe 12 ft. in diameter:

$$M_{0.5\pi r} = 0.0072 \times \tfrac{1}{2} \times 62.5 \times 6^3 \times 12 = 585 \text{ in-lb.}$$

For a $\frac{5}{16}$-in. shell:

$$f_s = \frac{585 \times 6}{\left(\dfrac{5}{16}\right)^2 \times 12} = 3\,000 \text{ lb. per sq. in.}$$

The value of T_1 is found next by substituting for $\dfrac{dS}{du}$ in Equation (61):

$$S = S_1 + S_2 = -\tfrac{1}{2}\, q\, r\, x\, [\sin u - 0.85 \sin 2\, u] \ldots\ldots\ldots (68)$$

and,

$$\frac{dS}{du} = -\tfrac{1}{2}\, q\, r\, x\, [\cos u - 1.70 \cos 2\, u] \ldots\ldots\ldots\ldots\ldots (69)$$

Then, by substitution in Equation (61), since $X = 0$,

$$T_1 = -\frac{1}{r}\int -\tfrac{1}{2}\, q\, r\, x\, (\cos u - 1.70 \cos 2\, u)\, dx + C_2$$

$$= \tfrac{1}{2}\, q\, \frac{x^2}{2}\, (\cos u - 1.70 \cos 2\, u) + C_2 \ldots\ldots\ldots\ldots\ldots (70)$$

For a simply supported pipe, with $x = \frac{L}{2}$, $T_1 = 0$, and Equation (70) becomes,

$$T_1 = \tfrac{1}{2}\, q\, (\cos u - 1.70 \cos 2\, u) \left(\frac{x^2}{2} - \frac{L^2}{8}\right) \ldots\ldots\ldots\ldots (71)$$

For $u = \pi$,

$$T_1 = -1.35\, q \left(\frac{x^2}{2} - \frac{L^2}{8}\right)$$

or 1.35 times the value derived for a pipe exactly full.

For continuously supported pipe:

$$T_1 = \tfrac{1}{2}\, q\, (\cos u - 1.70 \cos 2\, u) \left(\frac{x^2}{2} - \frac{L^2}{24}\right) \ldots\ldots\ldots\ldots (72)$$

Mr. Karlsson has also given some measurements for the deformations due to ring stresses for the old Kungfors pipe line in Sweden.[21] If the pipe when empty is considered to have zero deformation, then the maximum deformation for the pipe half full at the center line of the span amounts to $\frac{1}{8}$ in. The pipe is built of $\frac{5}{16}$-in plate with no stiffeners between supporting rings and has a diameter of about 9 ft. and is 38 ft. between the supports. This condition indicates clearly that a direct stress condition exists, practically, for this grade of filling since a possible ring moment of any consequence would cause considerably more deflection.

According to Mr. Samsioe,[22] pipe lines of $\frac{1}{4}$-in. plate, 12.5 ft. in diameter, and 43 ft. span and with no stiffeners between the supporting rings, are common practice in Sweden.

Rim Stresses.—An approximate value of f_{bm} for unyielding ring support can be found in the following manner. The author's notations are used

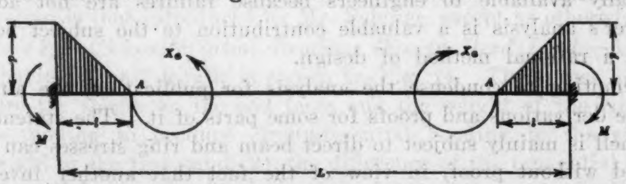

FIG. 13.

and l (Fig. 13) is the distance along the pipe axis from the center line of the ring support to the point, A, where the pipe shell practically has expanded an amount, r'_p, and, consequently, all the load is taken by the ring stresses.

[21] *Schweizerische Bauzeitung,* September 2, 1922.

[22] By letter to the writer.

It is obvious that the moment at this point, A, must have a value between zero and the value, X_a, which makes the tangent to the elastic line of the pipe shell in A parallel to the pipe axis. It is also evident that the load causing the rim stresses is equal to p at the ring support; and from this on it is decreasing and becomes practically equal to zero at Point A and remains very close to zero between the two points, A.

Assume a straight-line variation of the load between the center line of the ring support and Points A. First, consider the case in which the moment at A is assumed equal to zero. Equating r'_p and the cantilever deflection:

$$\frac{\frac{1}{2}\, p\, l^4}{15\, E\, I} = \frac{p\, r^2}{E\, t}$$

or,

$$l = 1.26 \sqrt{r\, t} \qquad (73)$$

$$M = 0.264\, p\, r\, t \qquad (74)$$

and,

$$f_{bm} = 1.58 \frac{p\, r}{t} \qquad (75)$$

In a similar way, for the case in which the moment at A is assumed equal to $X_a = \frac{1}{24}\, p\, l^2$, and,

$$f_{bm} = \frac{2.58\, p r}{t} \qquad (76)$$

Taking the average of the two values in Equations (75) and (76):

$$f_{bm} = 2.08 \frac{p r}{t} \qquad (77)$$

or 14% more than the value found by the author.

DONALD E. LARSON,[23] JUN. AM. SOC. C. E. (by letter).[23a]—There has been a noticeable scarcity of data for use in the design of large pipe lines. Something can be learned from records of past failures, but such information is not generally available to engineers because failures are not advertised. Mr. Schorer's analysis is a valuable contribution to the subject because it introduces a rational method of design.

In attempting to condense the analysis for publication, the author has omitted the derivations and proofs for some parts of it. The statement that the pipe shell is mainly subject to direct beam and ring stresses can scarcely be accepted without proof, in view of the fact that another investigator, Professor Raymond J. Roark, of the University of Wisconsin, reports circumferential bending stresses[24] in the shells of pipe lines having supports of

[23] Asst. Research Engr., Chicago Bridge & Iron Works, Chicago, Ill.

[23a] Received by the Secretary October 30, 1931.

[24] "A Study of Circumferential Bending of Pipes and Cylindrical Containers", *Bulletin No. 69*, Eng. Experiment Station Series, Univ. of Wisconsin, Madison, Wis.

the same type as those discussed by the author. Furthermore, the results of the two studies should be in agreement because both investigators used the same distribution of longitudinal horizontal shearing stress.

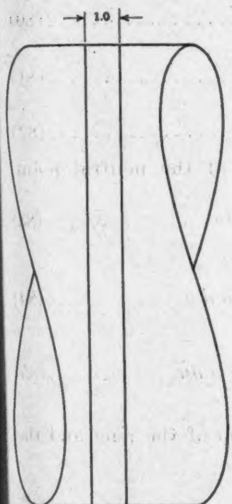

FIG. 14.

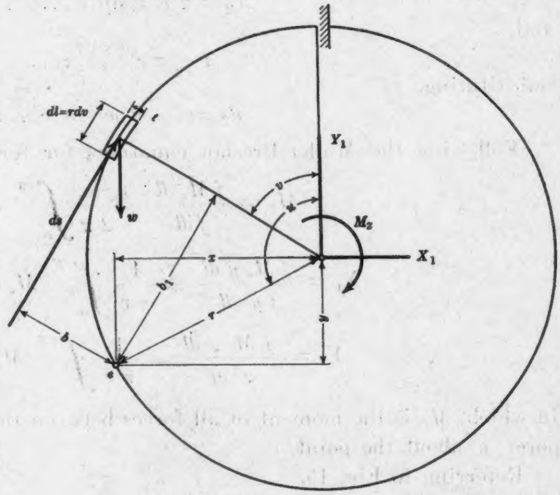

FIG. 15.

The writer has made a study to determine the reason for the disagreement in regard to circumferential bending stresses in the shells of pipes subject to dead load and full water load. An analysis is submitted as proof for the following statements:

(1) There are no circumferential bending stresses in the pipe shell under dead load if the shearing stresses act tangentially as indicated by the author.

(2) There are no circumferential bending stresses in the pipe shell under full water load if the shearing stresses act tangentially.

(3) The shearing stresses do act tangentially as indicated in the author's analysis.

The notation is that introduced by Mr. Schorer, except as otherwise given.

Example (1).—Circumferential Bending Due to Dead Load.—A portion of a pipe having a unit length is shown in Fig. 14. The forces acting on this ring are the shears on the end faces and the weight of the ring itself. The forces tending to produce circumferential bending in the ring are: (1) The weight of the ring; and (2), the difference of the tangential shearing forces on the end faces. Fig. 15 shows these forces acting on a small element of the ring. The difference in tangential shearing forces is given by the following equations:

$$ds = \frac{A\bar{y}\,dQ}{2I} \quad \text{.......................(78)}$$

in which, ds is the tangential shear per unit of circumference, due to change in total shear, dQ. In this case,

$$dQ = 2\pi\, r\, w \qquad (79)$$

$$A\bar{y} = 2\, r^2\, t \sin v \qquad (80)$$

and,

$$I = \pi\, r^3\, t \qquad (81)$$

Substituting,

$$ds = -\, 2\, w \sin v \qquad (82)$$

Following the Müller-Breslau equations for forces at the neutral point,

$$M_z = \frac{\int M_e\, dl}{\int dl} = \frac{1}{2\,\pi} \int_0^{2\pi} M_e\, du \qquad (83)$$

$$X_1 = \frac{\int M_e\, y\, dl}{\int y^2\, dl} = \frac{1}{\pi\, r} \int_0^{2\pi} M_e \cos u\, du \qquad (84)$$

$$Y_1 = \frac{\int M_e\, x\, dl}{x^2\, dl} = \frac{1}{\pi\, r} \int_0^{2\pi} M_e \sin u\, du \qquad (85)$$

in which, M_e is the moment of all forces between the top of the ring and the point, e, about the point, e.

Referring to Fig. 15,

$$M_e = -\int_0^u ds\; b\; r\; dv + \int_0^u w \sin v\; b\; r\; dv - \int_0^u w \cos v\; b_1\; r\; dv \qquad (86)$$

and,

$$b_1 = r \sin (u - v) = r\, (\sin u \cos v - \cos u \sin v) \qquad (87)$$

$$b = r\, [1 - \cos (u - v)] = r\, (1 - \cos u \cos v - \sin u \sin v) \qquad (88)$$

Substituting these values of b_1 and b in Equation (86) and performing the indicated integration,

$$M_e = -\, w\, r^2\, (1 - \cos u) \qquad (89)$$

Substituting this value of M_e in Equations (83), (84), and (85), and integrating between the limits, 0 and 2π,

$$M_z = -\, w\, r^2 \qquad (90)$$

$$X_1 = +\, w\, r \qquad (91)$$

$$Y_1 = 0 \qquad (92)$$

The moment in the ring at any point is,

$$M = -\, M_z - X_1\, r \cos u + M_e \qquad (93)$$

Substituting the values of M_z, X_1, and M_e from Equations (90), (91), and (89), respectively,

$$M = w\, r^2 - w\, r^2 \cos u - w\, r^2\, (1 - \cos u) = 0 \qquad (94)$$

This indicates that there is no circumferential bending in the shell of a truly circular pipe due to dead load, providing the shearing forces act tangentially as shown in Fig. 15.

Example (2).—Circumferential Bending Due to Water Load.—Fig. 16 shows the water-load forces tending to produce circumferential bending in the ring. These are: (1) The outward forces due to water pressure; and (2)

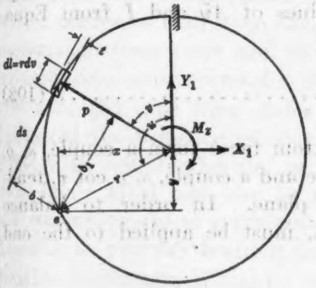

FIG. 16.

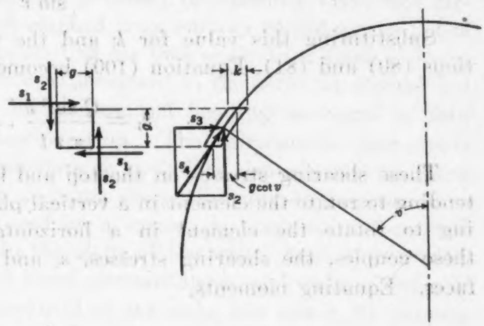

FIG. 17.

the difference between tangential shearing forces on the end faces. In this case,

$$dQ = q\,\pi\,r^2 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(95)$$

Substituting this value in Equation (78),

$$ds = -\,q\,r\,\sin v \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(96)$$

The outward force at any point is given by the relation,

$$p = qr\,(1 - \cos v) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(97)$$

The moment at the point, e, of all the forces between the top of the ring and e is,

$$M_e = -\int_0^u ds\; b\; r\; dv + \int_0^u p\; b_1\; r\; dv \ldots\ldots\ldots\ldots(98)$$

Substituting the values of s, b, p, and b_1 from Equations (96), (88), (97), and (87), respectively, and integrating between the limits, 0 and u,

$$M_e = 0 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(99)$$

Therefore, $M_z = 0$, $X_1 = 0$, and $Y_1 = 0$, and the moment in the ring at any point is equal to zero. This proves that there is no circumferential bending in the pipe shell due to full water load, providing the shearing stresses act tangentially as shown in Fig. 16.

Example (3).—Tangential Shearing Stresses.—It now remains to demonstrate that the shearing forces do act tangentially. It is in this respect that the author's analysis differs from that of Professor Roark. Fig. 17 shows an element of the pipe shell having a length, g, and a vertical height, g, such that the area of each of the four faces is equal to unity. The shearing stress on the horizontal faces is given by the relation,

$$s_1 = \frac{Q\,A\,\bar{y}}{2\,I\,k} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(100)$$

in which, s_1 is the unit horizontal shearing stress and,

$$k = \frac{t}{\sin v} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (101)$$

Substituting this value for k and the values of $A\bar{y}$ and I from Equations (80) and (81), Equation (100) becomes,

$$s_1 = \frac{Q \sin^2 v}{\pi \, r \, t} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (102)$$

These shearing stresses on the top and bottom faces form a couple, $s_1 \, g$, tending to rotate the element in a vertical plane and a couple, $s_1 \, g \cot v$, tending to rotate the element in a horizontal plane. In order to balance these couples, the shearing stresses, s_2 and s_3, must be applied to the end faces. Equating moments,

$$s_2 = \frac{s_1 \, g}{g} = s_1 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (103)$$

$$s_3 = \frac{s_1 \, g \cot v}{g} = s_1 \cot v \quad \ldots\ldots\ldots\ldots\ldots\ldots (104)$$

The resultant of s_2 and s_3 acts tangentially and has a magnitude,

$$s_4 = \sqrt{s_2^2 = v_3^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (105)$$

in which, s_4 is the tangential unit shear on the end faces.

Substituting the values of s_2 and s_3 given by Equation (100) and Equation (101),

$$s_4 = s_1 \sqrt{1 + \cot^2 v} = \frac{s_1}{\sin v} \quad \ldots\ldots\ldots\ldots\ldots\ldots (106)$$

Substituting the value of s_1 given by Equation (99),

$$s_4 = \frac{Q \sin v}{\pi \, r \, t} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (107)$$

This indicates that the shearing forces shown by the author are correct in both magnitude and direction.

The foregoing analysis has been based on the assumption that the horizontal longitudinal shearing stresses in the pipe vary as they would in a cylindrical beam. The author has not given the derivation of the differential equation from which this distribution of shearing stresses was obtained, but it has been shown that such a distribution results in a system of forces which hold each element of the shell in static equilibrium without the application of circumferential bending moments. Such a system of forces does a minimum amount of work and it may be argued from the principle of least work that the assumed distribution of horizontal longitudinal shearing stresses is the correct one. This being the case, there can be no doubt about the correctness of the author's statement that the pipe shell is mainly subject to direct beam and ring stresses.

RAYMOND J. ROARK,[25] ESQ, (by letter).[25a]—Because of work the writer had done some time ago on stresses in pipes and cylindrical containers, he is especially interested in Mr. Schorer's valuable paper. The conclusions to which the analysis of such stresses by the theory of elasticity leads, are surprising, and at variance with those reached from cursory consideration. The writer confesses with some chagrin that in his own analysis of these stresses, undertaken some years ago, an error with respect to the shearing stresses led to erroneous expressions for the circumferential bending moments in thin shells of the type here under consideration. The calculations then made indicated the presence of such moments in cylinders either partly or quite filled with liquid, whereas, as Mr. Schorer shows, the uneven pressure in a completely filled pipe can be resisted by a system of shearing and normal stresses which are at every point in the plane of the wall. As a consequence, there is no primary bending under these circumstances, and even a perfectly flexible diaphragm, if rigidly constrained at the ends, will retain its circular form. This fact first came to the writer's attention through an article by Thoma,[26] who shows, in essentially the same manner as Mr. Schorer, that not only in the case of a circular cylinder such as a pipe, but in the case of any developable surface, a distributed pressure can be resisted without primary bending stress.

The demonstration for the full vessel is clear and convincing, but it would appear that for a partly filled vessel some circumferential bending must occur. The writer has not had an opportunity to see the paper by Mr. A. F. Samisoe to which Mr. Schorer refers, and Mr. Schorer does not state whether this analysis of the half-filled cylinder indicates the existence of bending or not; but that the pressure in any partly filled vessel cannot be resisted by stresses which are at every point in the plane of the shell would seem to be sufficiently proved by the following.

Let f_u, f_x, and τ denote, respectively, the unit stress in circumferential tension, longitudinal tension, and tangential shear; let u and x, respectively, denote distance measured circumferentially and longitudinally from any chosen origin; let p, t, and r, respectively, denote the unit pressure, thickness of the shell, and radius of the cylinder. Then, if the assumption of plane stress is correct, the following equations must apply:

$$f_u = \frac{p\,r}{t} \qquad (108)$$

$$\frac{\partial f_u}{\partial u} + \frac{\partial \tau}{\partial x} = 0 \qquad (109)$$

$$\frac{\partial f_x}{\partial x} + \frac{\partial \tau}{\partial u} = 0 \qquad (110)$$

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[25] Associate Prof. of Mechanics, Univ. of Wisconsin, Madison, Wis.

[25a] Received by the Secretary November 6, 1931.

[26] "Die Beanspruchung freitragender gefüllten, Rohre durch das Gewicht der Flussigkeit," *Zeitschrift für das gesamte Turbinenwesen*, February 20, 1920. The discussion is extended by Schwerin in a paper "Uber die Spannungen in freitragender gefüllten Rohren," *Zeitschrift für angewandte Mathematik und Mechanik*, Vol. II, October, 1922.

Equation (108) shows that for any point above the water line, $f_u = 0$; hence, $\frac{\partial f_u}{\partial u} = 0$ in that region, and it follows from Equation (109) that $\frac{\partial \tau}{\partial x} = 0$ and that, since $\tau = 0$ at, say, the middle of a cylinder supported at the ends, it must be zero everywhere above the water-line; but this is obviously inconsistent with the assumption of plane sections and ordinary beam action of the cylinder as a whole.

It seems, therefore, that in the partly filled cylinder there must be some circumferential bending. An analysis of this circumferential bending may be made, as follows:[27]

Let a segment of the pipe of unit length be divided by a vertical diameter; the forces acting on the resulting half-ring are as shown in Fig. 18. They comprise: (1) The unknown couples, C_1 and C_2; (2) the unknown tensions, F_1 and F_2; (3) the water pressure which, at any point at an angular distance, α, from the bottom of the ring, amounts to $p = q\,r\,(\cos\alpha - \cos\theta)$ lb. per in. of arc (q being the specific weight of the contained liquid, in pounds per cubic inch); and (4) the tangential shearing stress which, at any point at an angular distance, α, from the bottom of the ring, amounts to

FIG. 18.—FORCES ON FREE RING (WEIGHT NEGLECTED).

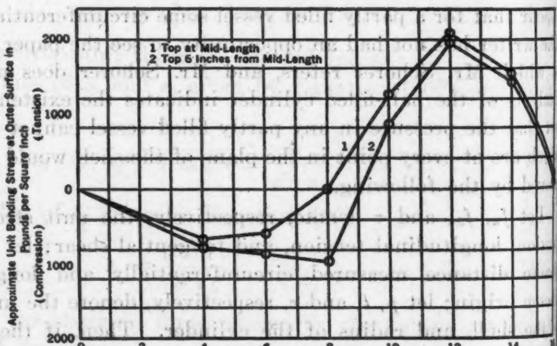


FIG. 19.—MEASURED CIRCUMFERENTIAL BENDING STRESSES ON HORIZONTAL CYLINDER.

$\tau_s\,t = \lambda\,q\,r \sin\alpha$ lb. per in. of arc (λ being the fractional degree of fullness of the cylinder). It is easy to show that the bending moment due to these forces at any point, A, at an angular distance, α', from the bottom of the ring is, for points below the water level ($\alpha' < \theta$):

$$M = C_1 - F_1\,r\,(1 - \cos\alpha') - \lambda\,q\,r^3\left(\tfrac{1}{2}\alpha' \sin\alpha' + \cos\alpha' - 1\right) + q\,r^3\left(\tfrac{1}{2}\alpha' \sin\alpha' - \cos\theta + \cos\theta\cos\alpha'\right) \ldots\ldots\ldots (111)$$

[27] For complete analysis, see *Bulletin No. 69*, Univ. of Wisconsin, "A Study of Circumferential Bending of Pipes and Cylindrical Containers." Some of the results therein presented are invalidated by the error, corrected in the present discussion, in the expression for the shear stress.

and for points above the water level ($\alpha' > \theta$), the moment is:

$$M = C_1 - F_1 r (1 - \cos \alpha') - \lambda q r^3 (\tfrac{1}{2} \alpha' \sin \alpha' + \cos \alpha' - 1)$$
$$+ q r^3 [\sin \alpha' (\tfrac{1}{2} \theta - \tfrac{1}{2} \sin \theta \cos \theta)$$
$$+ \cos \alpha' (\cos \theta - \cos^2 \theta - \tfrac{1}{2} \sin^2 \theta] \ldots\ldots\ldots\ldots (112)$$

If the shell is assumed to be perfectly flexible, $C_1 = 0$ and $F_1 = q r^2 (1 - \cos \theta)$. If these values are submitted in Equations (111) and (112), they vanish for $\theta = 180°$; that is, for the case of the completely filled vessel, but do not vanish for any smaller value of θ, showing that for the partly filled vessel some bending must exist.

The existence of the couples, C_1 and C_2, is thus shown to be necessary for equilibrium, and C_1, C_2, F_1, and F_2, all become indeterminate. They may be found from the conditions of statics and the additional relations:

$$\int_0^{2\pi} M r \, d\alpha' = 0 \ldots\ldots\ldots\ldots (113)$$

$$\int_0^{2\pi} M r^2 (1 - \cos \alpha') \, d\alpha = 0 \ldots\ldots\ldots\ldots (114)$$

which equations simply express the facts, obvious from symmetry, that as between the bottom and the top of the ring there is neither lateral nor angular displacement. Evaluation of these integrals and solution of the resulting equations gives:

$$F_1 = \frac{2 q r^2}{\pi} \left[\tfrac{3}{8} \sin \theta \cos \theta - \tfrac{1}{8} \theta - \frac{\pi}{2} \cos \theta - \frac{\pi}{2} \cos^2 \theta + \frac{\pi}{4} \sin^2 \theta - \tfrac{1}{4} \theta \cos^2 \theta \right]$$
$$+ \tfrac{3}{4} q \lambda r^2 \ldots\ldots\ldots\ldots (115)$$

and,

$$C_1 = \frac{2 q r^3}{\pi} \Big[\tfrac{5}{8} \sin \theta \cos \theta - \tfrac{3}{8} \theta - \frac{\pi}{2} \cos \theta + \frac{\pi}{2} \cos^2 \theta + \frac{\pi}{4} \sin^2 \theta - \tfrac{1}{4} \theta \cos^2 \theta$$
$$- \tfrac{1}{4} \sin \theta + \tfrac{1}{2} \theta \cos \theta - \tfrac{1}{4} \sin \theta \cos^2 \theta - \tfrac{1}{4} \sin^3 \theta \Big] + \tfrac{1}{4} q \lambda r^3 \ldots (116)$$

When these values of C_1 and F_1 are substituted in Equations (111) and (112), general expressions are derived, that make it possible to calculate the bending moment at any point for any degree of fullness of the cylinder. Obviously, such calculations must be made with the greatest precision if the results are to have any value, because the expression for M represents the algebraic sum of a large number of nearly equal terms, and even a small error in these individual terms will produce a large relative error in the result.

In order to secure a verification of the main points of the foregoing discussion, namely, that there is no circumferential bending in the full cylinder, and that there is such bending in the partly filled cylinder, the writer made some tests which, although of a rather "rough and ready" nature, may be of interest. The results are shown in Fig. 19.

A specimen was constructed which consisted essentially of a circular pipe with closed ends; the diameter was 16 in.; the length, 8 ft.; the thickness

of the shell, which was galvanized steel, was 0.02 in.; and the thickness of the square heads, which were riveted and soldered to the shell, was 0.125 in. Figs. 20 and 21 represent photographs of this specimen. It was intended to secure as thin, long, and flexible a shell as practicable, and it is apparent that the proportions are much more extreme in this respect than were those of the pipe described by Mr. Schorer, the ratio of diameter to thickness being 800 as against 44, and the ratio of unsupported length to diameter being 6 as against 3.7.

This specimen was tested in two ways. First, while in the horizontal position as shown in Fig. 20, the weight being carried by the edges of the end plates, it was gradually filled with water. A deflectometer of 4-in. span, provided with an Ames dial reading to 0.0001 in., was placed transversely on top of the vessel near mid-length, to detect and to measure approximately any circumferential bending that took place. The readings secured, of course, do not afford an accurate indication of the bending stresses produced, but if it is assumed that the bending of the 4-in. arc over which the deflectometer reached, was intermediate in form between that of a beam with end couples and that of a beam with end supports and center load, the stresses corresponding to a given deflection could be easily computed, and should represent roughly the order of magnitude of the actual bending stresses produced. Fig. 19 represents stresses thus computed, plotted against the degree of fullness of the cylinder. One curve represents results secured by measurements at mid-length, the other by measurements 6 in. from mid-length; the two sets of readings were taken to eliminate in some degree the effect of irregularities in form of the shell—flat spots, dimples, etc.—almost unavoidable in such a thin plate.

Second, in order to simulate conditions in a stand-pipe or tank during an earthquake shock, one end was cut off the specimen, which was then bolted, at the other end, to a heavy timber platform. It was then placed in a vertical position, filled with water, and inclined to the vertical as shown in Fig. 21. The deflectometer was used to detect any bending that might occur at the upper end, but even when inclined as steeply as 9° to the vertical the bending was practically negligible. This degree of inclination corresponds roughly to a horizontal acceleration of 5.1 ft. per sec. per sec., which is in excess of the acceleration ordinarily assumed in design when earthquake shock is taken into account. By raising and lowering one end of the supporting platform rapidly, the tank was caused to oscillate with sufficient violence to result in water being thrown out over the top, and yet no appreciable distortion of the cross-section resulted. No effort was made to ascertain accurately the acceleration produced, but from a rough observation of the amplitude and period it was judged to be at least 4 or 5 ft. per sec. per sec.

Although these tests were manifestly crude and the results unquestionably influenced by geometrical irregularities in the specimen and the very rough method used for measuring the stresses, they seem to show none the less definitely that circumferential bending is absent in the case of a filled vessel, and is present, but ordinarily negligible, in the case of a partly filled

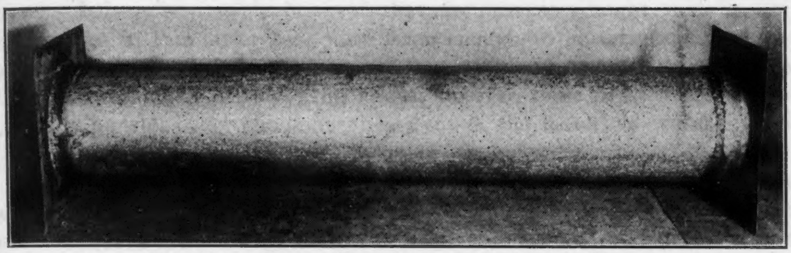


FIG. 20.—CIRCULAR TEST PIPE, WITH SQUARE HEADS.

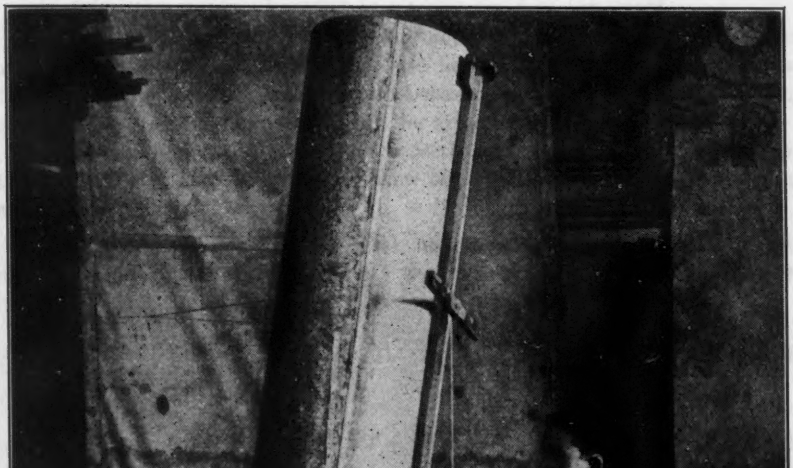
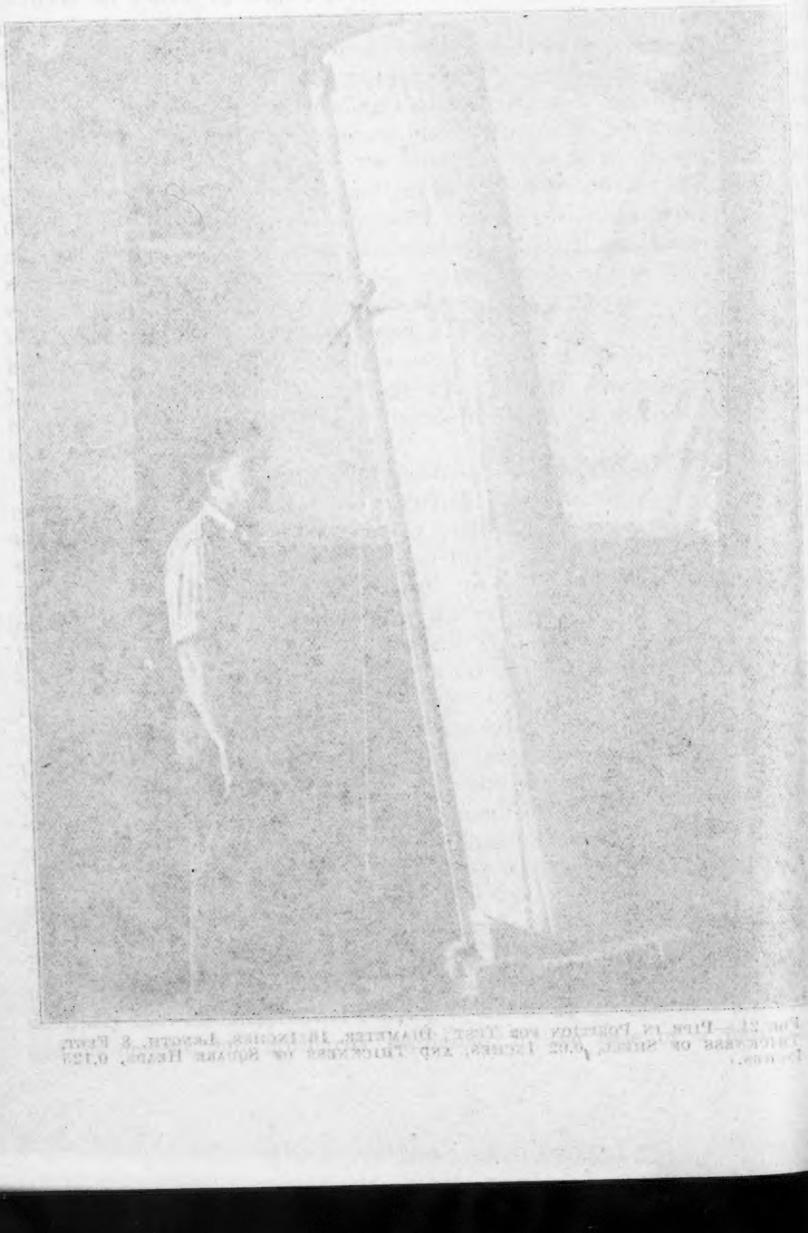


FIG. 21.—PIPE IN POSITION FOR TEST; DIAMETER, 16 INCHES, LENGTH, 8 FEET, THICKNESS OF SHELL, 0.02 INCHES, AND THICKNESS OF SQUARE HEADS, 0.125 INCHES.



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vessel. It is quite possible, however, that circumstances might occur in which the stresses in this latter case were large enough to be serious in the event of frequent reversal, as in the case of a paper digester or other rotating vessel. For pipes and tanks of usual proportions, however, these stresses would almost certainly be well below the endurance limit of the materials ordinarily used.

F. W. HANNA,[28] M. AM. SOC. C. E. (by letter).[28a]—The author has presented a useful technical analysis of the design of large pipe lines. The writer has checked this analysis with much interest, and agrees in general with the methods and results obtained.

The following comments apply to specific parts of the paper:

(1) In Equations (38) and (40) the moment, M_s, is negative rather than positive, as given by the author, because he has elsewhere assumed counter-clockwise moments to be negative and because the signs of moments are not determinable from the signs of either the arms or the forces involved.

(2) Equation (39) seems to be incorrectly stated and Equation (37) seems to be incorrectly used in the derivation of Equation (41). The writer suggests the following development.

Using the author's diagrams and nomenclature, the load on the pipe carried to one disk due to the pipe's weight and the water's weight for the condition when the pipe is filled just to the top,

$$Q = [\pi (r + t)^2 - \pi r^2] \frac{w L}{2} + q \pi r^2 \frac{L}{2} \quad \ldots\ldots\ldots\ldots (117)$$

or,

$$Q = \frac{\pi L}{2} [(2 r t + t^2) (w + q r^2)] \quad \ldots\ldots\ldots\ldots (118)$$

Since t^2 is small compared with $2 r t$, it may be dropped without serious error; then, multiplying by $\frac{\sin v}{\pi r}$,

$$\frac{Q \sin v}{\pi r} = \frac{L}{2} (2 t w + q r) \sin v \quad \ldots\ldots\ldots\ldots (119)$$

Applying the author's Equations (5) and (9) to Fig. 6 for the angle, v,

$$s = \frac{L}{2} (2 t w + q r) \sin v \quad \ldots\ldots\ldots\ldots (120)$$

Equating the values of the right members of Equations (119) and (120),

$$s = \frac{Q}{\pi r} \sin v \quad \ldots\ldots\ldots\ldots (121)$$

Since counter-clockwise moments are assumed to be negative, the moment, M_s, of the shearing force, s, is,

$$M_s = - \int_0^u b\, s\, r\, dv \quad \ldots\ldots\ldots\ldots (122)$$

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[28] Chf. Engr. and Gen. Mgr., East Bay Municipal Utility Dist., Oakland, Calif.

[28a] Received by the Secretary, October 30, 1931.

Also, from Fig. 6, it is clear that,

$$\cos (u - v) = \frac{r - b}{R} \dots\dots\dots\dots\dots\dots\dots (123)$$

Transposing, solving for b, and expanding $\cos (u - v)$ in Equation (123),

$$b = r - R (\sin u \sin v + \cos u \cos v) \dots\dots \dots\dots (124)$$

Substituting the value of b from Equation (124) and the value of s from Equation (121) in Equation (122), and re-arranging,

$$M_s = \frac{Q R}{\pi} \int_0^u (\sin u \sin^2 v + \cos u \cos v \sin v - \frac{r}{R} \sin v) \, dv \dots (125)$$

Intergrating Equation (125) between the limits of u and 0, the following final value of M_s is obtained,

$$M_s = \frac{Q R}{\pi} \left[\frac{u \sin u}{2} - \frac{r}{R} (1 - \cos u) \right] \dots\dots\dots\dots (126)$$

Thus, it is apparent that the error in the author's value for b in Equation (39) and his omission of the minus sign before the right-hand member of Equation (38) have cancelled one another and given the correct value of M_s in his Equation (41). The remaining results based on Equation (41) are apparently correct.

The author has developed equations for finding the total normal and bending stresses in the circular supporting disk to moment only. There are stresses due also to thrust and shear, and while those due to shear are generally very small, those due to thrust may be quite considerable. The total normal bending and shearing stresses involving the effect of rib-shortening, moment, and shear, may be found by the writer's method of arch dam analysis.[29]

In short, this method involves the principle of deflections through the application of a rigid bracket between the free end of the cut section and the elastic center. Redundant forces at this point replace the released forces, and also the dummy unit loads at the elastic center. The deflection formula when complete contains terms that provide: (1) For deflection due to rib-shortening; (2) for that due to bending moment; (3) for that due to shear; and (4) for that due to volume changes. For the problem under discussion the third and fourth terms of the formula may be omitted, but the first two terms should be used for accuracy. The author has used only the second term, that providing for deflection due to bending moments.

[29] "The Design of Dams," by Messrs. Hanna and Kennedy, p. 227.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

SOIL MECHANICS RESEARCH

Discussion

By Messrs. J. C. Meem, and H. de B. Parsons

J. C. Meem,[19] M. Am. Soc. C. E. (by letter).[19a]—This paper is a contribu-
ion of unquestioned value to the profession, particularly if or when the urther research in connection with foundations and lateral earth pressures hall have been carried out.

The writer believes that it is practicable to establish definitely, through ractical tests for:

(*a*) The actual subsidence of various soils under varying loads at various depths; and

(*b*) The actual lateral pressure of soils at varying depths.

In order to conduct such tests so that the results will be of definite prac-
ical value, it will be necessary first, as to Test (*a*), to re-arrange tables for he so-called bearing values of soils so that they will show the expected settle-
ment or subsidence under loading rather than the load which such soils may be xpected to carry; that is, the writer advocates making a simple classification f soils, such as loam, clay, sand, gravel, and rock, with combinations of each, nd testing them:

(*c*) Superficially to loads of 1, 2, 4, and 8 tons;

(*d*) At depths of 10 to 20 ft., unconfined to loads of 4, 8, 10, and 20 tons; and,

(*e*) At depths of 20 ft., or more, confined (as at the base of piles or small shafts) to loads of 20, 40, 60, and 80 tons per sq. ft.

The subsidence under the various loads should be noted in each case. The ests should be made:

(*f*) On normally dry soils; and,

(*g*) On saturated soils.

With proper co-ordination and co-operation the results of tests made in ccordance with an established procedure could be collected from widely vary-
ng sources and tabulated; and, eventually, tables of real practical value, owing the subsidence of soils under loadings, could be permanently stablished.

Note.—The paper by Glennon Gilboy, Jun. Am. Soc. C. E., was presented at the meeting of the Structural Division, Boston, Mass., October 10, 1929, and published in October, 1931, *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

[19] Cons. Engr., Brooklyn, N. Y.

[19a] Received by the Secretary November 12, 1931.

In connection with Test (*b*), the apparatus for testing lateral pressure, the writer is convinced that the use of the apparatus described in the paper will mark an important step in the advancement of soil testing, but that it does not go far enough.

He is convinced that the most effective apparatus for making such tests would be a box, rigidly braced externally and having four or more sliding bulkheads confining one side; that is, in a box of a 20-ft. cube, there should be four superimposed bulkheads, 5 by 20 ft., to confine the soil on one face. These bulkheads or louvers should be of beam or box-girder construction, having arms extended and designed to slide independently of each other on cross-arms or tracks extended beyond the box. Four adjustable weights to each louver operating through pulleys and cords attached to the extending arms of the louvers, should be arranged to offset or balance the sliding friction, so that light pressure applied at any point in the louver face will cause it to move. Then, by filling the box with kiln-dried sand, and increasing the inside weights to balance the pressure of the soil, the exact pressure on each independent louver can be determined.

The tests should be made with dry sand or dry disintegrated materials only, such as kiln-dried sand, pea gravel, ordinary gravel, broken stone, and, if desired, wheat, and buckshot. In the writer's judgment, any attempt to make these tests on soils or materials which are cohesive, or are made so by moisture, will be abortive and misleading. It will be a simple matter to co-relate the results of the tests on the dry materials with those having cohesive properties, by observing or measuring and comparing the "break" lines of each.

The writer believes that if exhaustive tests are made on the dry soils, as noted, they will establish conclusively the fact that arching properties and lateral pressures in soils are definitely measurable; and that they will further confirm the fact that the pressure of soil due to its lateral transmission is non-accumulative vertically, as in tunnels, although it increases laterally in direct ratio to the depth, being always at a maximum at a point slightly above the center of the vertical face.

In suggesting further procedure, which the writer feels may be of value, he does not wish to imply any lack of his appreciation of the paper itself, which is of great interest and value to the profession.

H. de B. Parsons,[20] M. Am. Soc. C. E., (by letter).[20a]—The writer, having read this interesting paper, would like to ask Professor Gilboy why temperature does not enter Equation (2), for finding permeability, or the quantity of water flowing through a soil? Perhaps the author had in mind experiments made in a laboratory where variations in temperature were small.

In a study, made by the writer, of water in a pervious soil in its natural state, the effect of temperature was very marked. In this study, the temperature range was about 30° Fahr. Furthermore, the formula used for determining the flow of water through filter sands shows that the velocity varies with the temperature.

[20] Prof. Emeritus, Rensselaer Polytechnic Inst.; Cons. Engr., New York, N. Y.

[20a] Received by the Secreary, October 19, 1931.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852

DISCUSSIONS

SPECIFICATIONS FOR BOUNDARY SURVEYS

PREPARED BY

COMMITTEE OF SURVEYING AND MAPPING DIVISION ON BOUNDARY SURVEYS

Discussion

BY H. FREDERIC PETERSON, ASSOC. M. AM. SOC. C. E.

H. FREDERIC PETERSON,[7] ASSOC. M. AM. SOC. C. E. (by letter).[7a]—The standard specifications for boundary surveys, as presented by the Committee, have stressed chiefly the benefits to be derived from their use by municipal surveys. It is logical to devote the largest share of attention to these, but due consideration should also be given to the surveys of the vast areas of land which were practically worthless, originally, but which have been found to be highly valuable.

As an example of the adaptability of the rectangular co-ordinate system to large areas, it might be of interest to outline the method used in mapping a triangulation system, covering about 1 000 sq. miles, including a portion of that immense oil deposit in California known as the Kettleman Hills Field. A rectangular co-ordinate plan was required to facilitate the detailed surveys which were to be made later. The primary triangulation system was carefully and accurately surveyed, and secondary stations were "three-pointed" to the primary points. The quadrangle formed by 1° of latitude and 1° of longitude was decided upon to be governed by one origin of rectangular co-ordinates, placed at the intersection of the center meridian and the center parallel of latitude of the quadrangle, with the basis of bearings taken as due north.

This unit contains an area of approximately 3 800 sq. miles, the longest distance from the origin being about 44 miles, which would give the plane co-ordinates at this extreme point an accuracy with respect to the origin of about 1 part in 18 000 or 20 000. Although this system allows a variation of the plane azimuths from geodetic azimuths of nearly 0° 20′ at extreme east-

NOTE.—The Specifications for Boundary Surveys, Prepared by the Committee of Surveying and Mapping Division on Boundary Surveys, was published in May, 1931, *Proceedings*. Discussion of the paper has appeared in *Proceedings*, as follows: September, 1931, by Messrs. George M. Bleekman, G. D. Whitmore, and William W. Michael; and November, 1931, by J. S. Doddo, M. Am. Soc. C. E.

[7] Acting Chf. Draftsman, Survey and Drafting Dept., Shell Oil Co., Los Angeles, Calif.

[7a] Received by the Secretary October 26, 1931.

erly or westerly boundaries of the quadrangle, corrected azimuths can be easily obtained if necessary.

One of the primary reasons for the adoption of this method was to provide for a uniform system intended eventually to include the vast amount of triangulation which had been previously surveyed, covering the entire San Joaquin Valley. Incidentally, by developing the primary triangulation to a scale of 10 000 ft. to the inch, it was conveniently mapped on two sheets, one containing the northern half and the other the southern half of the one-degree quadrangle, while the details of the secondary system were mapped at 3 000 ft. to the inch on drawings of the same size.

Many of the property line surveys in the San Joaquin Valley were retracements of surveys originally made between 1850 and 1880. The record monuments were quite frequently obliterated and the surveyor continually faced the problems of locating section corners by proportion. The Manual of Instructions for the Survey of the Public Lands of the United States, issued by the Department of the Interior, proved invaluable as the official guide in all cases.

The solution offered in the Manual for the restoration of lost section corners, however, often raised questions by the field men. The lines run through the trial points located proportionately on the latitudinal and longitudinal lines between found original corners are, according to the Manual, to be run east or west and north or south to locate the position of the restored corner. The advisability of the use of cardinal directions from the trial points in all such cases has often been questioned by surveyors, as this rule results in the location of the restored corner being dependent upon the bearings of the lines between found original corners. Two independent surveys can easily result in a large divergence of bearings, depending upon the bases used by the respective surveyors.

It would not necessarily be a violation of the basic principle for the control of the latitudinal and longitudinal position of the lost corner, mentioned in the Manual, to suggest that another interpretation might seem more adaptable. Inasmuch as the position of the restored corner is only an approximation at best, the elimination of any single factor permitting any possible difference in the case of two or more independent retracements would be a step toward consistency. By instructing the surveyor to turn at right angles to the trial lines at the proportional points to locate the restored corner, any difference in its location by other retracements would occur only through differences of chaining of the trial lines and not through the possible adoption of different bases of bearings. This method would also have the added advantage of being a purely mechanical field process, tending to eliminate possible errors in field computations.

In Fig. 1, purposely exaggerated for clarity, *A*, *B*, *C*, and *D* are found original corners. Point *F* is the proportional point on the trial line, *AB*, while Point *E* is the proportional point on the trial line, *CD*; Point *G* is the intersection of the lines turned at right angles to the trial lines; Point *H* is the intersection of a line turned east through *F* with a line turned south through *E*. It is obvious that *H* is dependent upon the bearings of *AB* and *CD*, while *G* can be located regardless of their bearings.

Assuming that Lines *CE* and *DE* were equal and Lines *AF* and *BF* were equal, by virtue of equal record distances, as very often would be the case in interior section lines, then *CG* would equal *DG* and *AG* would equal *BG*,

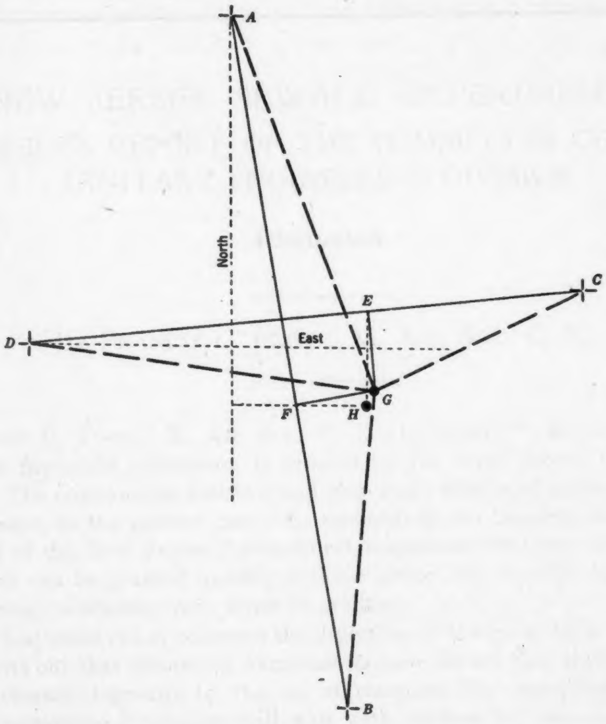

FIG. 1.

which would be consistent with the record. With lines connecting *H* to *A*, *B*, *C*, and *D*, however, this would not be true, and the proportioning would be inconsistent with the record.

The suggestion of the Committee that prospective purchasers of real property should insist on viewing the monuments limiting the boundaries thereof is very well taken. A little different phase of the same suggestion would be for owners of section property to insist on being shown the original monuments at the corners of their properties, or the monuments by which their corners are proportioned. In one case in mind, the owner of several sections of possibly valuable oil lands became quite irate on learning that a survey of his lands by a county surveyor for the purpose of fencing it, was incorrect. The surveyors in charge of the retracement of the land boundaries for the oil company showed him the original charcoal corners and witness marks and explained to him their significance. The actual property lines were about 200 ft. distant from his fence lines.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852

DISCUSSIONS

NEW JERSEY SEWAGE EXPERIMENTS

PROGRESS REPORT OF THE COMMITTEE OF THE SANITARY ENGINEERING DIVISION

Discussion

BY FRANCIS C. FOOTE, M. AM. SOC. C. E.

FRANCIS C. FOOTE,[3] M. AM. SOC. C. E. (by letter).[3a]—In reviewing this report, a favorable impression is created by the form chosen by the Committee. The outstanding features and important results of sanitary engineering research, in the present case—the research of the Department of Sewage Disposal of the New Jersey Agricultural Expriment Station—are given in a form that can be grasped quickly without going into lengthy detail and yet, unlike many abstracts, very little is omitted.

The first subdivision concerns the digestion of sludge at high temperatures and points out that laboratory experiments have shown that time of digestion can be shortened greatly by the use of thermophillic organisms. The Sanitary Engineering Profession will wait with interest for the results obtained when the principles determined in the laboratory are applied on a plant scale in an actual installation. The last section of the report directs attention to an instance in which the gas generated during digestion at 84° Fahr., when used as a source of heat, was sufficient to maintain the digestion temperature. Questions naturally arising are: Will this fact be true with thermophillic digestion? will expensive insulation be necessary? and will auxiliary heat be required?

Relative to the test of the value of crushed stone, slag, and gravel as filter media, the statement is made that "there was not much difference in the 'over-all efficiencies' produced by the various beds over a period of eight months, as shown by the results in Table 3." Using various media, Table 3 shows a percentage of reduction in bio-chemical oxygen demand by

NOTE.—This report of the Committee of the Sanitary Engineering Division on New Jersey Sewage Experiments was presented at the meeting of the Sanitary Engineering Division at Norfolk, Va., on April 16, 1931, and published in August, 1931, *Proceedings*. Discussion of the report has appeared in *Proceedings*, as follows: November, 1931, by Edmund B. Besselievre, M. Am. Soc. C. E.

[3] Senior Asst. Engr., Morris Knowles, Inc., Pittsburgh, Pa.

[3a] Received by the Secretary November 4, 1931.

trickling filters, of 82 for crushed stone, 75 for slag, and 77 for gravel. These figures, indicating an advantage in favor of crushed stone, were apparently obtained from Table 8 of the 1930 Annual Report of the Department of Sewage Disposal of the New Jersey Agricultural Experiment Station. This table is based on "two single groups of analyses." Furthermore, a check of the mathematics of Table 8 indicates that the 82% for crushed stone should be 75 per cent. With this correction, the percentage removals are practically the same, rather than indicating an advantage in favor of crushed stone.

The data for Table 3 of the report should perhaps have been taken from Table 2 of the New Jersey report, which shows the average results during the entire eight months covered by the experiments. The average 5-day biochemical oxygen demand of the influent to all the filters was 173 parts per million. The average bio-chemical oxygen demand of the effluents was 103 parts per million for crushed stone, 80 parts per million for slag, and 109 parts per million for gravel. These results show purification efficiencies for the trickling filters alone of 40% for crushed stone, 54% for slag, and 37% for gravel. The results under discussion indicate that slag rather than crushed stone is best from the standpoint of B. O. D. removal. The following is the statement of the New Jersey report on the question (page 28):

"Although the results obtained over eight months warrant some general statements, it is too early to come to definite conclusions in respect to the efficiency of the different filter media. There is no doubt that from a chemical standpoint the slag was performing best, followed by crushed stone and gravel. From a bacteriological standpoint, the results seem to indicate that all stone media may be equally efficient in *B. coli* removal."

The conclusions reached, upon the basis of the studies made, are of extreme interest, but definite findings should perhaps be reserved until a greater number of experiments have been made under varying conditions and at several localities.

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DISCUSSIONS

HIGHWAY CONSTRUCTION MANAGEMENT

Discussion

BY CHESTER MUELLER, ASSOC. M. AM. SOC. C. E.

CHESTER MUELLER,[5] ASSOC. M. AM. SOC. C. E. (by letter).[5a]—This paper forcefully calls attention to the avoidable waste prevalent in the highway construction industry and the efforts of the U. S. Bureau of Public Roads to reduce it. As shown by Mr. Allen, this branch of engineering is in need of, and is entitled to, the painstaking form of research heretofore confined to the realms of pure science and only recently applied to manufacturing processes by the Mechanical Engineering Profession through the media of time-studies.

A few comments on some of the features of the construction industry that were treated by the paper might not be amiss, particularly if they are examined in the light of municipal paving projects.

Policies.—Wisely, Mr. Allen foregoes a lengthy discussion of policy. While "economic necessity and priority" are proper topics for engineering consideration, nevertheless, a pavement is frequently constructed at the insistence of an organized group of citizens—when the cost is not to be locally assessed—in spite of the lack of necessity for it. Likewise, a much needed pavement may be opposed by abutting property owners when such improvement is to be paid for by assessment against such owners. Preparation of data clearly will be expected of an engineer, but where local sentiment or the pocketbook is involved, the exercise of well-founded engineering judgment will be denied him.

Contracts.—The paper, dealing essentially with the execution of work, implies that an eingineer must do his share to foster efficient methods. He can do this in the very beginning by making the time of letting a paving contract seasonable. Receiving bids during the period of uncertain markets, for work under adverse weather conditions, and with unreasonable time clauses, will certainly affect bid prices and will have considerable bearing on subsequent construction administration.

NOTE.—The paper by T. Warren Allen, M. Am. Soc. C. E., was presented at the meeting of the Highway Division, New York, N. Y., January 16, 1930, and published in September, 1931, *Proceedings*. Discussion of the paper has appeared in *Proceedings*, as follows: November, 1931, by A. E. Holcomb, M. Am. Soc. C. E.

[5] Prin. Asst. Engr., Dept. of Public Affairs, Newark, N. J.

[5a] Received by the Secretary, October 28, 1931.

By making the "Information to Bidders" clear and complete, the contractor may better formulate his plan of operation. Adequate plans of the work will also aid better planning.

It is possible that, even though they are poorly executed, the foregoing items may not prevent efficient administration; but if they are properly constituted, they will certainly be conducive to it.

Subsurface Work for Utility Companies.—In cities, probably the largest single problem handicapping successful administration, is the subsurface work that public utility companies perform in advance of paving. Such work is essential, and in order to reduce the disruption of traffic to a minimum, it often becomes necessary for a paving contractor to begin his operations while underground work is also in process. In co-ordinating this work and in expediting the subsurface work the engineer can do much to obtain for the public and the contractor a paving job efficient in time and in cost. Subsurface work by utility companies in the public streets and highways certainly tends to resist efficient highway construction administration, and in the extensive field of endeavor which it covers, much is yet to be found to aid the paving industry.

Specifications.—Engineers are generally prone to let existing specifications do service from year to year without any changes other than the addition of clauses occasionally, as local experience dictates.

Specifications should be kept modern by elimination and alteration as well as by addition. In other words, complete revision may be desirable after existing specifications are scrutinized in the light of recent researches and practices. As Mr. Allen states clauses that require a mixing time for concrete in excess of the time actually required with modern equipment, result in wasteful administration. In some communities and localities, clauses of that nature may persist in specifications for years after they have been proved obsolete by competent investigators.

Design.—Failure to consider the utility of roads and approaches, and the sequence of contracts, as illustrated in the paper, is indicative of poor design and thwarts efficient administration. The writer does not fully subscribe, however, to the author's suggestion that paving design should attempt to provide for the 100% economic use of the contractor's equipment. In establishing lines and grades it should be only a minor influence since in most present-day designs the cost to the public of operating vehicles over a road after the improvement is completed, is as important as the initial construction cost. In the illustration given by Mr. Allen, the average loss of about $46.63 per day, due to irregular requirements for hauling units, may add about 4 cents per sq. yd. to the cost of the pavement, whereas changes in grade may cost far more than that, if not in construction, possibly in the operating cost of the vehicular traffic using the road. A design permitting a practically uniform daily number of hauling units and the efficient use of a contractor's equipment, is, of course, highly desirable provided other essential elements are not sacrificed. To determine to what extent the design of a road can be

influenced, without adding to its cost, requires considerable additional research, and the efforts of the Bureau of Public Roads in this direction will be most helpful.

If the provision of suitably located disposal areas is considered as a function of design and as the task of the engineer rather than that of the contractor, then the design of the improvement, in so far as these areas are concerned, is most vitally associated with efficient administration. Most contracts leave the question of disposal of excess earth to the contractors, which is one reason why an alert contractor who plans his work can under-bid his competitors.

Equipment.—Equipment is given an important place by Mr. Allen, a place that it well deserves. Serviceable equipment with readily available spare parts, used and maintained in a proper fashion, determines more than any other factor, the speed and smoothness with which a paving job can be completed. The forceful and convincing sales data of leading equipment manufacturers prove to be a boon to contractors in forcing them to keep abreast of the times.

Personnel.—Many contractors are short-sighted with respect to the employment of labor on the maintenance of approach and utility roads. Use of men in what is sometimes referred to as "non-productive" work frequently has a direct bearing on the efficiency with which a contractor can operate his equipment and almost always indirectly contributes to it.

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Founded November 5, 1852

DISCUSSIONS

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[5] Prin. Asst. Engr., Dept. of Public Affairs, Newark, N. J.

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AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
DISCUSSIONS

CONSTRUCTION WORK ON A FEDERAL RECLAMATION PROJECT

Discussion

BY JOHN SANFORD PECK, ASSOC. M. AM. SOC. C. E.

JOHN SANFORD PECK,[5] ASSOC. M. AM. SOC. C. E. (by letter).[5a]—In reading this paper, the writer's interest was captured primarily by the sections devoted to the description of the methods used to insure high quality concrete. The success of the project, from a service standpoint, depended entirely upon the quality of the concrete produced on the job. The greatest possible skill in hydraulic design would have been nullified entirely if the concrete had failed under the extremely severe conditions of service to which it was exposed. The results obtained, as judged by the test cylinders taken from the job, are another vindication of scientific methods of concrete manufacture.

The successful results that were obtained depended entirely on the degree of knowledge and skill of the concrete inspectors, and the author's description of the "Instructions for Concrete Inspectors" that were prepared for use on this project, together with the tables that he mentions in detail, point to a handbook of more than passing interest. It must have been successful in teaching the essential features of the control of the water-cement ratio on the job to practical-minded men to whom anything resembling theory is anathema.

With reference to the use of an admixture of diatomaceous earth in the concrete, the writer would like to know what the reasons were for using an admixture at all, and just how much this admixture contributed to the workable properties of the concrete? A description of the laboratory technique used in determining the need for the admixture and in actually measuring the increase in the workability secured by using it, would be of great interest and value.

For some time the writer has been of the opinion that, while admixtures in the proper proportions do no harm to the concrete, they do not appreciably

NOTE.—The paper by Morris Mason, Jun. Am. Soc. C. E., was published in October, 1931, *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

[5] Instr. in Civ. Eng., Coll of the City of New York, New York, N. Y.

[5a] Received by the Secretary November 5, 1931.

improve a mix, except perhaps to add an indefinite and unmeasurable quality of "fatness," which can only be detected by the eye. The results of several series of laboratory tests made by the writer have confirmed this opinion and have tended to show that the best and cheapest admixture is an increase in the cement of from 5 to 10 per cent.

This part of Mr. Mason's paper again shows the results that can be obtained by scientific methods of concrete control, and it is worth noting that a paper[8] by William M. Hall, M. Am. Soc. C. E., also discusses the practical use in the field of the results that have been obtained and thoroughly proved in the laboratory.

[8] "Manufacturing Concrete of Uniform Quality", by William M. Hall, M. Am. Soc. C. E., *Proceedings*, Am. Soc. C. E., May, 1931, p. 675.

PROCEEDINGS

of the

American Society

of

Civil Engineers

(INSTITUTED 1852)

PAPERS AND DISCUSSIONS

VOL. LVII

JANUARY TO DECEMBER, 1931

ALBANY, N. Y.
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1931

VOL. LVII

SUBJECT INDEX

AUTHOR INDEX

APPLICATIONS FOR ADMISSION AND FOR TRANSFER

The Constitution provides that the Board of Direction shall elect or reject all applicants for *Admission* or for *Transfer,* and, in order to determine justly the eligibility of each candidate, the Board must depend largely upon the Membership for information.

This list is issued to members in every grade for the purpose of securing all such available information, and every member is urged to scan carefully each monthly list of candidates and to furnish the Board with data in regard to any applicant which may aid in determining his eligibility. It is the *Duty* of all *Members* to the *Profession* to assist the Board in this manner.

It is especially urged, in communications concerning applicants, that errors in the record be pointed out and a *Definite Recommendation as to the Proper Grading in Each Case* be given, inasmuch as the grading must be based upon the opinions of those who know the applicant personally, as well as upon the nature and extent of his professional experience. If facts exist derogatory to the personal character or to the professional reputation of an applicant, they should be promptly communicated to the Board. *Communications Relating to Applicants are considered by the Board as Strictly Confidential.*

The Board of Direction will not consider the applications herein contained from residents of North America until the expiration of thirty (30) days, and from non-residents of North America until the expiration of ninety (90) days from December 15, 1931.

MINIMUM REQUIREMENTS FOR ADMISSION

Grade	General Requirement	Age	Length of Active Practice	Responsible charge of work
Member	Qualified to design as well as to direct work	35 years	12 years*	5 years
Associate Member	Qualified to direct work	27 years	8 years*	1 year
Junior	Qualified for sub-professional work	20 years†	4 years*	
Affiliate	Qualified by scientific acquirements or practical experience to co-operate with engineers	35 years	12 years*	5 years
Fellow	Contributor to the permanent funds of the Society			

* Graduation from a school of engineering of recognized reputation is equivalent to 4 years active practice.

† Membership ceases at age of 33 unless transferred to higher grade.

LIST OF APPLICANTS.

Names and Addresses of Applicants for Admission and for Transfer on this List, Arranged Alphabetically.

The fact that applicants give the names of certain members as references does not necessarily mean that such members endorse.

The number in the center above each record indicates the serial number of the applicant for the current year, and that at the left the district in which he resides.

The abbreviations in Italics represent, respectively, *TT*, Total Time; *SP*, Sub-Professional Work; *P*, Professional Work; *RC*, Responsible Charge; *D*, Design. The figure for Total Time is determined by adding one-half the time spent in Sub-Professional Work to the time spent in Professional Work. The figures showing the amount of time spent in Responsible Charge and on Design are the estimate of the Applicant. The allowance of four years for graduation or of one-half of a year for each academic year successfully completed in an engineering college without graduation is included in Total Time and Professional Work.

FOR ADMISSION

1230

(9) ANDEREGG, RUPERT ANDREW, 3607 Bellecrest Ave., Cincinnati, Ohio. (Age 37. Born Kasota, Minn.) 1919 C. E., Univ. of Cin. *TT 4: P 4.*—June 1912 to Sept. 1917 (while student) Chainman, Rodman and Instrumentman on surveys (1 5/6 years), on railroad trackwork and concrete viaduct construction (4 months), Engr. and Asst. Supt. on simplex pile construction (6 months) and Inspector on concrete building erection (1 year).—Sept. 1917 to Feb. 1919 with U. S. Army, 1 year in foreign service with 1st Engrs. and at Headquarters, 92d Div., as Asst. Div. Gas Officer, 2d Lieut., Engrs., and 1st Lieut., Chemical Warfare Service. *TT 1.4: P 1.4: RC 1.4.*—June 1919 to July 1921 Supt. of Constr., The Texas Co., Port Arthur, Tex., on layout and supervision of pressure (24 units) and crude (8 units) stills, steam power house, pump house, tanks, office building, etc. ($7 000 000). *TT 2.1: P 2.1: RC 2.1.*—Sept. 1921 to date with Civ. Eng. Dept., Univ. of Cincinnati, until Sept. 1923 as Instructor, Sept. 1923 to Jan. 1929 and Sept. 1929 to June 1930 Asst. Prof., and since June 1930 Associate Prof. of Civ. Eng.; Jan. to Sept. 1929 (on leave of absence) with Cincinnati U. S. Dist. Engr's office, in charge of Flood Control Sec., carrying on power, flood, navigation and irrigation investigations on Miami, Licking and Kentucky Rivers, writing preliminary reports on Kentucky and Miami Rivers, planning and executing topographic surveys, stream gauging, collecting hydrologic data, selecting dam and reservoir sites, plans and cost estimate for dams, locks, power houses, etc., power studies at various dam sites, etc., *TT 10.2: P 10.2 RC 8.2.*—*TT 17.7: P 17.7: RC 11.7.* Refers to J. M. Belknap, W. L. Glazier, C. E. Hammell, L. S. Johnson, H. H. Kranz, H. B. Luther, F. L. Raschig, R. W. Renn, E. K. Ruth, H. Schneider, W. S. Winn.

1231

(4) ANNICK, WILLIAM ALFRED, 1010 Jackson St., Scranton, Pa. (Age 26, Born Scranton, Pa.) 1923 to 1926 Draftsman, Pennsylvania Dept. of Highways. *TT 1.5: SP 1.5.*—1926 to date with Bureau of Eng., Scranton, Pa., until 1930 as Draftsman, and since then Levelman; also on private work, assisting First Asst City Engr. of Scranton, on laying out lots, preparing land subdivisions, making surveys, general construction, etc., and assisting Borough Engr. of Dunmore, Pa., on pavement and sewer plans. *TT 2.5: SP 2.5.*—*TT 4: SP 4.* Refers to C. H. Buckius, A. B. Cohen, R. D. Richardson, C. M. Roberts, T. R. Williams, F. G. Wolfe.

1232

(1) BAUMGART, EMILIO HENRIQUE, Caixa Postal 922, Rio de Janeiro, Brazil. (Age 42. Born Blumenau, Santa Catharina, Brazil.) 1910 B. of Sc. and Letters, Gymnazio Santa Catharina, 1919 C. E., Pol. School, Univ. of Rio de Janeiro.—1911 to 1922 with L. Ried-

linger, Gen. Contr., 3 years as Surveyor on hydro-electric power developments, 3 years as Draftsman, Designer and Estimator, detailing reinforced concrete, etc., and 6 years Constr. Supervisor and Chf. Designer in charge of main office, closely connected with about 200 jobs, including buildings, reinforced concrete bridges, retaining walls, dams, piers, etc. *TT 10.5: SP 1.5: P 9: RC 6: D 9.* 1923 to date in private practice as Civ. and Cons. Engr., 2 years on design and construction of over sixty industrial, commercial and private buildings in Rio de Janeiro, and since 1925 giving engineering advice to architects, constructors, manufacturers, Federal Govt. Depts., etc., on design, details and inspection of 350 reinforced concrete structures of all types. *TT 8: P 8: RC 8: D 8*—*TT 18.5: SP 1.5: P 17: RC 14: D 17.* Refers to J. M. Bell, A. F. de Lima Campos, J. F. Partridge, C. de S. Rabello, T. A. Ramos, W. T. Webb.

1233

(1) BETTS, FRANKLIN PERLEY, 65½ Poningo St., Port Chester, N. Y. (Age 22. Born Port Chester, N. Y.) 1931 B. S. in C. E., N. Y. Univ. *TT 4: P 4.* Refers to A. Haring, C. T. Schwarze

1234

(2) BLANCHARD, WILLIAM LEONARD, 49 South Ave., Melrose, Mass. (Age 49. Born Chelsea, Mass.) 1908 B. S. in Civ. Eng., Univ. of Vt. *TT 4: P 4.*—May 1908 to Jan. 1910 Transitman, City of Chelsea, Mass. *TT 0.8: SP 0.8.*—Jan. to May 1910 Tech. Asst. to C. A. Allen, Massachusetts Inst. of Technology, compiling engineers' handbook. *TT 0.2: SP 0.2.*—May 1910 to Nov. 1917 Transitman, Inspector and Draftsman, Boston Elev. Ry. Co., on surveys, line and grade construction for foundations and steel structures, car-houses, track layouts, etc.; over 3½ years Asst. Engr. and Res. Engr. on Charles River Viaduct, North Station changes, Guild St. and Alfred St. shops, Mystic River and Malden River bridges. *TT 5.6: SP 2; P 3.6: RC 3.6.*—Nov. 1917 to May 1918 Asst. Engr. on shipways construction, Hog Island, Pa. *TT 0.6: P 0.6.*—May 1918 to date Res. Engr. and Supt. of Constr., Crandall Eng. Co., on railway dry dock, Charleston Navy Yard, dry dock for Lake Champlain Transportation & Dry Dock Co., Winnisimmitt dry dock, Chelsea, Mass., College Point, Union Shipyard and Elizabeth City Shipyard dry docks, Morton and Davies, Quebec, dry docks, Neponset River bridge and Edison Illuminating Co. syphons under three bridges at Charleston North, Chelsea North, Chelsea South and Warren bridges, Green Pond bridge, Falmouth, Mass., and Gloucester outfall sewer; also Res. Engr. in charge of Sandusky Port and Rochester Port surveys, etc.; work included estimating. *TT 13.5: P 13.5: RC 13.5.*—*TT 24.7: SP 3: P 21.7: RC 17.1.* Refers to C. H. Brown, E. A. Fisher, W. F. Pond, C. A. Poole, H. C. Willcox.

1235

(11) BROWNE, HAROLD GEORGE, Y. M. C. A. Hollywood, Cal. (Age 23. Born Brooklyn, N. Y.) 1930 B. S. in Civ. Eng., and 1931 C. E., Princeton Univ. *TT 4: P 4.*—Summer 1928 with Ewalde Asphalt Paving Co., New York City.—*TT 4: P 4.* Refers to G. E. Beggs, F. H. Constant, F. C. Noble.

1236

(9) BRYSON, CARL ARCHIBALD, U. S. Engr. Office, Marietta, Ohio. (Age 22. Born Columbus, Ohio.) 1931 B. C. E., Ohio State Univ. *TT 4: P 4.*—Summer 1928 Asst. Engr. Div. 6, Ohio State Dept. of Highways, Columbus, Ohio, acting as Inspector and Layout Engr. on grade-separation project.—*TT 4: P 4.* Refers to E. F. Coddington, J. M. Montz, C. T. Morris, J. C. Prior, C. E. Sherman, R. C Sloane.

1237

(7) BUDD, JOHN MARSHALL, 11 Summit Court, St. Paul, Minn. (Age 24. Born Des Moines, Iowa.) 1930 B. S., Sheffield Sci. School, Yale Univ. *TT 4: P 4.*—June to Oct. 1930 member of party studying and reporting on railroads in U. S. S. R. (under Ralph Budd). *TT 0.3: P 0.3.*—Nov. 1930 to date Asst. to Elec. Engr., Great Northern Ry. Co., on construction of steam power plants. *TT 1: P 1.*—*TT 5.3: P 5.3.* Refers to R. Budd, H. K. Dougan, J. C. Tracy.

1238

(16) BURTON, THOMAS EDWIN, 1260 Plass Ave., Topeka, Kans. (Age 50. Born Grandview, Ill.) 1903 B. S., and 1918 C. E., Wichita Univ.—June 1903 to April 1917 on construction work for James W. Burton and other road and building contractors.—April 1917 to July 1919 Asst. County Engr., Sedgwick County, Kans., on surveys and plans for Federal Aid bridge and road paving projects, and county improvements. *TT 1.3: P 1.3: D 1.3.*—July 1919 to date with Kansas State Highway Comm., until July 1925 as Res. Engr. in Sedgwick County, in charge of construction of Federal Aid road and

bridge work (about 50 miles) and county road and bridge work (4 miles), then Chf. of Plans, in charge of drafting room and of checking all plans for county and state road and bridge work, also prepared estimates and supplemented specifications and since March 1929 Div. Engr., First Dist. in complete charge of construction, maintenance and administration of state road and bridge work in 1st Div. (1700 miles). *TT 12.4: P 12.4: RC 12.4: D 12.4.*—*TT 13.7: P 13.7: RC 12.4: D 13.7.* Refers to A. A. Anderson, W. V. Buck, O. J. Eidmann, F. W. Epps, C. I. Felps, M. W. Watson.

1239

(1) CASSIDY, GEORGE FRANCIS, 318 East One Hundred Thirty-sixth St., New York City. (Age 25. Born New York City.) Student, Cooper Union Evening School (1926-1928 and 1929-1930) and Columbia Univ. Extension (1928-1929 and 1931). March to Nov. 1927 Transitman, Foundation Co., New York City, assisting Field Engr. on construction work. *TT 0.2: SP 0.2.*—Jan. 1928-Jan. 1931 Surveyman, U. S. Engr. Office, New York City, in field and office, contract work and tidal studies, after Aug. 1929 in charge of field tidal observations and computations. *TT 3.1: P 3.1.*—Feb. 1931 to date Eng. Asst., Board of Transportation, New York City, laying out, inspecting, checking, etc., of contract construction work. *TT 0.8: P 0.8.*—*TT 4.1: SP 0.2: P 3.9.* Refers to J. M. Buckley. A. E. Clark, C. C. Covert, W. N. Harper, P. Sander.

1240

(1) CHRISTIANSEN, RAYMOND SEVERIN, 558 Broad Ave., Ridgefield, N. J. (Age 26. Born East Hartford, Conn.) 1926 Certificate in Architectural Constr., Pratt Inst., Brooklyn, N. Y.—Sept. 1922 to Oct. 1924 and during school vacations Rodman, Chainman, Instrumentman, Draftsman, etc. with C. Henry Olmsted, C.E. *TT 1: SP 1*—June 1926 to May 1928 Draftsman and (approx. 1 year) Chf. Draftsman and Supt. with Beineix & Snyder, Archts., New York City, on alterations, etc., involving field study. *TT 1.4: SP 0.5: P 0.9: RC 0.9*—May 1928 to July 1931 Draftsman, Designer, etc. with W. W. Chapin, New York City, on steel, reinforced concrete and wood structures. *TT 2: SP 1.2: P 0.8: D 0.8.*—July to Sept. 1931 with Ball & Snyder, Cons. Engrs. as Pile Inspector on Rikers Island Prison. *TT 0.2: P 0.2: RC 0.2*—*TT 4.6: SP 2.7: P 1.9: RC 1.1: D 0.8.* Refers to J. Bourguignon, W. W. Chapin, A. L. P. Johnson, F. O. Price, H. H. Snyder.

1241

(13) CLAPHAM, FRANCIS JOSEPH, 901 Grizzly Peak Blvd., Berkeley, Cal. (Age 35. Born Mt. Vernon, S. Dak.) Aug. 1919 to Jan. 1922 and Aug. 1923 to June 1924 student, Univ. of California. *TT 1.5: P 1.5.*—Jan. 1922 to Aug. 1923 Workman or Constr. Foreman, on building construction, reinforced concrete, structural steel, frame, etc., buildings and bridges. *TT 1: SP 0.5: P 0.5: RC 0.5.*—June 1924 to July 1929 Draftsman and Designer with H J. Brunnier, San Francisco, on design of office, class-room and industrial buildings, hotels, and many other types of structures; 1 year also student, Univ. of California. *TT 4.7: SP 0.2: P 4.5: RC 4.5: D 3.5.*—July 1929 to May 1930 Asst. to Supt. of construction on Shell Oil Bldg. in San Francisco. *TT 0.8: P 0.8: RC 0.8.*—May 1930 to April 1931 Structural Engr., Shell Oil Co., San Francisco, designing structures for Shell Chemical Plant at Pittsburg, Cal. *TT 1: P 1: RC 1: D 1.*—April to June 1931 Structural Engr., Mills Estate Co., on design of alteration for Mills Bldg. in San Francisco. *TT 0.2: P 0.2: RC 0.2: D 0.2.*—July 1931 to date Structural Engr. for N. B. Ellery on design of post-office buildings for Stockton and Sacramento, Cal. *TT 0.4: P 0.4: RC 0.4: D 0.4.*—*TT 9.6: SP 0.7: P 8.9: RC 7.3: D 5.1.* Refers to H. J. Brunnier, E. L. Cope, W. S. Hassenmiller, C. J. Nobmann, J. Skytte.

1242

(4) COHEN, CHARLES, 6213 Delancey St., Philadelphia, Pa. (Age 25. Born Philadelphia, Pa.) 1927 diploma in Mun. Eng., Drexel Inst. Evening Diploma School.—July 1924 to date with Dept. of City Transit, Philadelphia, until May 1930 Chainman, Survey Corps Asst. on base line surveys, Rodman, Instrumentman and Party Leader on construction of Broad St. Subway, calculating payment estimates being in charge of calculating estimates on two contracts; May to Sept. 1930 Jun. Asst. Engr., being Office Engr. in charge of estimates, progress and record plans and field layout on construction of concourse in Market St., Philadelphia, and since Sept. 1930 Asst. Engr., being Asst. Sec. Engr. on construction of Locust St. Subway, in charge of field corps and layout, estimates and office data, inspectors and construction of 2 000 ft. of subway. *TT 6.1: SP 1.5: P 4.6*. Refers to J. L. Dodge, L. B. Manley, C. E. Myers, R. C. Scott, C. H. Stevens.

1243

(5) COLE, LUKE ADOLPHUS, 26 V Street N. W., Washington, D. C. (Age 58. Born Parkersburg, W. Va.) Sept. 1902 to date with Div. of Tides and Currents, U. S. Coast and Geodetic Survey, until Oct. 1918 as Clerk, then Tidal Mathematician, on reduction of soundings, tabulation, reduction and analysis of tidal observations, prediction of tides for annual tide tables, inspection and leveling at tide stations, and preparing tide notes for charts; since March 1925 has been Chf., Sec. of Datum Planes, in responsible charge of determination of basic tidal datum planes, preparing technical publications relative to tidal bench marks and datum planes and instructions for field parties for proper distribution of tide stations used in connection with hydrographic surveys. *TT 24: SP 5: P 19: RC 6.7.* Refers to F. S. Borden, W. Bowie, L. P. Disney, H. A. Marmer, P. C. Whitney.

1244

(1) DAVILA, FEDERICO, Maracaibo, Venezuela. (Age 28. Born Cabo Rojo, Porto Rico.) Registered Civ. Engr. in Porto Rico.—1926 B.S. in C.E., Tri-State Coll.—Summer 1924 and 1924-1925 (while student) on surveys, etc., for Ricardo Skerret, Civ. Engr. and Contr., of Mayaguez, Porto Rico, and others.—Summer 1925 Instrumentman, Gifford Constr. Co., Jamaica, N. Y.—June 1926–Dec. 1927 Instrumentman, Schaefer Eng. Corporation, Long Beach, N. Y., on construction of jetties, concrete pavements, sewers and water-supply lines, surveying, computing drafting, etc. *TT 1.2: SP 0.2: P 1: RC 0.5.*—May to July 1928 Jun. Engr., New Jersey State Highway Comm., Jersey City, N. J., on Lincoln Highway Viaduct, giving grades and checking road surface for pavements, layout of curves on adjacent approaches. *TT 0.1: SP 0.1.*—July to Oct. 1928 Instrumentman, Montauk Beach (N. Y.) Development Corporation, dredging, surveys, location of transmission power lines, drafting, etc. *TT 0.1: SP 0.1.*—Oct. 1928 to date with Lago Petroleum Corporation, Maracaibo, Venezuela, topographical drafting of tank farms, Instrumentman in charge of grading for 43 tanks, drainage and railroad layout, at Punta Tamares (5 months), Asst. Cost Engr., analyzing cost reports, keeping records of actual field construction costs, and checking estimates for proposed work (about 9 months), Surveyor, Lago properties, including sites for gasoline service stations in Maracaibo (5 months), in charge of finishing construction of two modern service stations in city (about 3 months), on survey and maps of water-supply system for Lago Terminal, Maracaibo, Inspector on construction jobs, including painting Manager's house and construction of filter galleries for lake water (6 months), and (past 9 months) Engr. in charge of construction of bulk loading station ($85 000) at Los Haticos, Maracaibo, including supervision of materials warehouse for the job. *TT 2.9: SP 0.1: P 2.8: RC 2.2.*—*TT 4.3: SP 0.5: P 3.8: RC 2.7.* Refers to J. O. Davis, A. A. Dedouloff, H. R. Gabriel, R. Kleppe, E. W. Silitch, J. A. Wahler, R. R. Wiggins.

1245

(1) DONATO, DANTE FRANK JOSEPH, 1816 White Plains Ave., New York City. (Age 24. Born New York City.) 1931 C. E., Manhattan Coll. *TT 4: P 4.*—June to Sept. 1931 Foreman, Scali Constr. Co., estimating, bidding, supervising excavation and foundation work and stone masonry for new buildings and generally responsible for work. *TT 0.2: P 0.2: RC 0.2*—*TT 4.2: P 4.2: RC 0.2.* Refers to J. J. Costa, D. C. Waite.

1246

(16) EDSTRAND, JOHN PHILIP, 3020 Forest Ave., Kansas City, Mo. (Age 30. Born Chicago, Ill.) 1930 B.S. in Civ. Eng, Armour Inst. Tech. *TT 4: P 4.*—June 1930 to date Jun. Engr., U. S. Engr. Office, Kansas City, Mo., on flood control, discharge, rainfall and flood damage studies, flood-control cost estimates, levee design, reports. *TT 1.4: P 1.4.*—*TT 5.4: P 5.4.* Refers to A. C. Bux, G. C. Haydon, J. O. Hunt, D. H. McCoskey, R. L. Stevens.

1247

(13) FLETCHER, PHILIP NELSON, 2532 Le Conte Ave., Berkeley, Cal. (Age 26. Born Peotone, Ill.) 1931 B. S., Univ. of Cal. *TT 4: P 4.*—Dec. 1923 to Feb. 1927 with McMillan & McMillan, San Jose, Cal., until May 1925 Rodman, Chainman and Draftsman, then Computer, Instrumentman and Chf. of Party on subdivision work. *TT 1.7 SP 1.4: P 0.3: RC 0.3.*—Feb. to Aug. 1927 with S. D. Balch, Los Gatos, Cal., as Concrete Inspector on water reservoirs. *TT 0.6: P 0.6: RC 0.6.*—Summers 1928, 1929 and 1930 with J. A. Love, San Francisco, Cal., successively as Field Engr. on high pressure gas line work, Installation Engr. on meter stations, and Constr. Engr. on river crossing, for high-pressure natural-gas lines.—June 1931 to date Asst. Engr. on model tests of proposed San Francisco-Oakland Bridge. *TT 0.4: P 0.4.*—*TT 6.7: SP 1.4: P 5.3: RC 0.9.* Refers to R. E. Davis, C. Derleth, Jr., F. S. Foote, C. G. Hyde, B. Jameyson, J. G. McMillan, W. B. McMillan.

1248

(14) GRIGSBY, KLINE BURDETT, 711 North West Twenty-sixth St., Oklahoma City, Okla. (Age 25. Born Blandinsville, Ill.) 1931 B.S. in C.E., Univ. of Okla. *TT 4: P 4.* Refers to J. F. Brookes, N. E. Wolfard.

1249

(1) GUPPY, JOHN WARREN, 165 Franklin St., Bloomfield, N. J. (Age 29. Born Marblehead, Mass.) 1923 B. S., Dartmouth Coll., 1924 C. E., Thayer School of Civ. Eng. *TT 4: P 4.*—May 1924 to date with American Bridge Co., until Nov. 1927 as Draftsman, Trenton (N. J.) Plant, detailing structural steel for bridges and buildings, then Asst. Engr., New York office, 10 months in Bldg. Dept., and remainder of time in Bridge Dept., estimating and designing structural steel for buildings and bridges. *TT 5.7: SP 1.7: P 4. RC 3.9: D 0.9.*—*TT 9.7: SP 1.7: P 8: RC 3.9: D 0.9.* Refers to J. E. Elliott, R. Fletcher, C. F. Goodrich, O. E. Hovey, C. Kendall, R. R. Marsden, S. J. Ott, B. B. Priest, A. P. Richmond, Jr.

1250

(13) HEAD, JAMES RAYMOND, 1100 Esplanade, Chico, Cal. (Age 44. Born Garden Grove, Cal.) Registered Civ. Engr., State of California.—Aug. 1906 to May 1907 student, Univ. of California. *TT 0.5: P 0.5.*—Aug. 1905 to Aug. 1906 and June 1912 to Sept. 1918 with Southern Pacific R.R., until Dec. 1907 as Rodman, and after June 1912 successively Draftsman, and Asst. Engr. at Bakersfield, San Joaquin Div. *TT 6.6: SP 0.6: P 6.*—May to Dec. 1907 Transitman, Union Lumber Co., Ft. Bragg, Cal., on railroad preliminary and location. *TT 0.3: SP 0.3.*—March 1908 to July 1911 with Arizona Power Co., Prescott, Ariz., on preliminary and construction of hydro-electric plant on Fossil Creek. *TT 2.3: SP 1: P 1.3: RC 1.3.*—July 1911 to June 1912 member of firm, Ensign Eng. Co., Los Angeles, Cal., general engineering. *TT 0.9: P 0.9: RC 0.9.*—Jan. 1919 to Dec. 1929 with Polk & Robinson, Civ. Engrs., Chico, Cal., on municipal, irrigation, drainage and general engineering work; also City Engr. of Chico. *TT 9.9: P 9.9: RC 5: D 2.*—Dec. 1929 to date with California Corrugated Culvert Co. of West Berkeley. *TT 2: P 2: RC 2: D 1.*—*TT 22.5: SP 1.9: P 20.6: RC 9.2: D 3.* Refers to R. L. Anderson, T. A. Bither, B. T. Hudspeth, C. F. Mau, A. C. Norton, F. S. Robinson, F. T. Robson, P. A. Welty.

1251

(13) HERKOMER, JOHN ANTHONY, Salt Springs, Martell, Cal. (Age 30. Born London, England.) May to Oct. 1922 and Sept. 1923 to May 1924 Chainman and Rodman, Nevada State Highway Dept., on road construction. *TT 0.6: SP 0.6.*—Sept. 1924 to date with Pacific Gas & Elec. Co., until June 1927 as Chainman, Rodman and Instrumentman, field work, also some office work and estimating, on Pit River Hydro-electric Projects No. 3 and No. 4; since June 1927 Instrumentman and Asst. Engr., on Mokelumne River Hydro-electric Project, field and office work on construction of dams, canals, tunnels, etc., surveys and estimating, design and layout of construction plants. *TT 6.6: SP 0.5: P 6.1: RC 2.*—*TT 7.2: SP 1.1.: P 6.1: RC 2.* Refers to W. Dreyer, G. C. Green, O. W. Peterson, R. D. Reeve, I. C. Steele.

1252

(7) HOLDAMPF, CARL RICHARD, 3465 North Humboldt Ave., Milwaukee, Wis. (Age 26. Born Little Rock, Ark.) Sept. 1925 to June 1927 student, Bldg. Design and Constr. (completed course), Wisconsin Univ., Extension Div.—Oct. 1923 to Jan. 1926 (after Sept. 1925 while student) Installer, Western Elec. Co., installing, assembling and testing telephone exchange equipment. *TT 1: SP 1.*—Feb. 1926 to date with C. S. Whitney, Milwaukee, Wis., until June 1927 (while student) as Draftsman and Computer and since June 1927 Civ. Engr. on designs and plans, surveys and reports for building frames, bridges, sewerage, water distribution, sewage plant and roads, including 1 year as Res. Engr. in charge of sewer construction. *TT 4.5: P 4.5: RC 3: D 3.5.*—*TT 5.5: SP 1: P 4.5: RC 3: D 3.5.* Refers to C. S. Gruetzmacher, D. W. Mead, W. R. Reuter, C. V. Seastone, C. N. Ward, C. S. Whitney.

1253

(13) HOLLIS, ROBERT WOOD, Jr., 2423 Blake St., Berkeley, Cal. (Age 23. Born Denver, Colo.) 1931 B. S., Coll. of Eng., Univ. of Cal. *TT 4: P 4.*—Summers 1926–1928 and Dec. 1928 to Jan. 1929 Chainman and Rodman with D. A. Loebenstein, San Diego and (1 month) Hussey & Belchers, Oakland, Cal. May to Aug. 1930, Dec. 1930 to Jan. 1931 and May to Oct. 1931 Surveyman, U. S. Engr. Office, Sacramento Dist., and South Pacific Div. Office, San Francisco, on water-supply investigation studies. *TT 0.2: P 0.2.*—Oct. 1931 to date Senior Eng. Office Aid, Div. of Highways, Dept. of Public Works, State of

California, preparing field survey notes for office and quantity calculations. *TT 0.1: SP 0.1.*—*TT 4.3: SP 0.1: P 4.2.* Refers to R. L. Egenhoff, H. G. Gerdes, C. I. Grimm, C. G. Hyde, R. S. Thomas.

1254

(10) INTEMANN, HENRY LUTHER, 1544 Adams St., Denver, Colo. (Age 25. Born New York City.) 1927 B. A. and 1929 B. S., Univ. of Colo. 1930 M. S., Univ. of Ill. *TT 4: P 4.*—June to Sept. 1930 Rodman, U. S. Bureau of Public Roads, Yellowstone Park, in field and office, making reports for Denver Office. *TT 0.2: P 0.2.*— Sept. 1930 to date Acoustical Engr., Rocky Mountain Celotex Co., Denver, Colo., correcting sound conditions in interiors, analyzed interiors, designed installations, estimated cost and made sales. *TT 1.2: P 1.2: RC 1.2.*—*TT 5.4: P 5.4: RC 1.2.* Refers to C. L. Eckel, W. C. Huntington.

1255

(1) JAKUBOWSKY, JOHN JOSEPH FELIX, 1376 East Twelfth St., Brooklyn, N. Y. (Age 21. Born New York City.) 1931 C. E., Rens. Pol. Inst. *TT 4: P 4.* Refers to L. W. Clark, H. B. Compton, T. R. Lawson, H. O. Sharp.

1256

(1) JEFFRIES, HERBERT OTTLEY, JR., Care, Banco Agricola Comercial, San Salvador, El Salvador. (Age 24. Born Panama City, Panama.) 1930 B. S. in C. E., Ga. School of Tech. *TT 4: P 4.*—Sept. 1924 to Aug. 1925 with Theo. H. Yale, Aldercreek, N. Y., as Timekeeper, Camp Boss, etc., on railroad construction International Rys. of Central America, San Salvador. *TT 0.5: SP 0.5.*—July 1930 to May 1931 with R. W. Hebard & Co. and R. Keilhauer, San Salvador, as Instrumentman to Res. Engr. on highways in Occidental Zone, Santa Ana, El Salvador, then Levelman and in charge of materials testing laboratory. *TT 0.9: P 0.9: RC 0.9.*—*TT 5.4: SP 0.5: P 4.9:* RC 0.9. Refers to R. P. Black, M. E. Gilmore, R. W. Hebard, M. E. Lopez Harrison, J. E. Mejia, W. R. Neel, F. C. Snow, L. F. Whitbeck, W. F. Wirth.

1257

(1) LANGBEIN, WALTER BASIL, 2385 Grand Concourse, New York City. (Age 24. Born Newark, N. J.) 1931 B. S. in C. E. Cooper Union (evening school). *TT 4: P 4.*—Aug. 1925 to date (until June 1931 while student) with Rosoff Subway Constr. Co., New York City, until March 1927 Rodman, on line and grades, then Eng. Asst., acting as Instrumentman and in office computing lines and grades and quantity estimates, and since July 1930 Constr. Engr., preparing plans, underpinning buildings, elevated railroad columns, design of temporary timber structures, overhead trestles, sheathing braces *TT 0.4: P 0.4: D 0.4.*—*TT 4.4: P 4.4: D 0.4.* Refers to A. H. Diamant, C. M. Madden, F. W. Stiefel.

1258

(7) LENZ, ARNO THOMAS, 108 Langdon St., Madison, Wis. (Age 25. Born Fond du Lac, Wis.) 1928 B. S., and 1930 M. S., in Civ. Eng., Univ. of Wis. *TT 4: P 4.*—Sept. 1928 to date Instructor in Hydr. and San. Eng., Univ. of Wisconsin; had charge of 1931 summer session; summers 1928-1930 with Nekoosa-Edwards Paper Co., Port Edwards, Wis., as Field Engr., being Inspector and Fieldman on building construction and (1930) Res. Engr. in charge of field work on installation and field design on 42-in. concrete pipe line (4 miles). *TT 3.3: P 3.3: RC 0.2.*—*TT 7.3: P 7.3: RC 0.2.* Refers to F. M. Dawson, D. W. Mead.

1259

(3) LOSEE, LAWRENCE KNICKERBOCKER, Upper Red Hook, N. Y. (Age 22. Born Upper Red Hook, N. Y.) 1931 B. Sc. in C. E., Rutgers Univ. *TT 4: P 4.*—Summer 1928 Inspector, State Highway Dept., on concrete highway construction.—*TT 4: P 4.* Refers to H. N. Lendall, H. M. Lewis, W. Rudolfs.

1260

(9) LUFF, WILLARD JOHN, 3046 Lincoln Blvd., Cleveland Heights, Ohio. (Age 30. Born Cleveland, Ohio.) 1922 B. S. in C. E., and 1928 C. E., Case School of Applied Sci. *TT 4: P 4.*—1922 to 1923 Civ. Engr. and Structural Designer, The Watson Eng. Co., general engineering design in connection with buildings and their location, etc. *TT 1.1: SP 0.1: P 1: D 1.*—1923 to date Cons. Engr. on development of installation methods and devices for building drainage, including research and design in development of fittings, co-operating with architects and engineers on plans and specifications for building drainage systems and with public officials in requirements for plumbing codes, development of methods for determining size of drainage and vent pipes in plumbing code for Dearborn, Mich., member of Bldg. Code Comm. of Cleveland Eng. Soc. to secure adoption of plumbing

codes on engineering principles, etc. *TT 8.2: P 8.2: RC 8.2: D 8.2.*—*TT 13.3: SP 0.1: P 13.2: RC 8.2: D 9.2.* Refers to R. L. Harding, J. J. Lucas, F. H. Neff, K. H. Osborn, W. C Parmley, R. B. Perrine, W. J. Watson.

1261

(13) McCREADY, HARRY LEWIS, 1543 Thirty-eighth St., Sacramento, Cal. (Age 49. Born Pierre, S. Dak.) 1914 A. B. in Civ. Eng., Stanford Univ. *TT 4: P 4.*—Dec. 1915 to May 1918 Asst. Hydr. Engr., State Reclamation Board of California, on flood-control project design and hydrographical work, Field Engr. on survey, and in charge of construction of, flood-control projects. *TT 2.3: P 2:3: RC 1.8: D 0.5.*—May 1918 to May 1922 Asst. Hydr. Engr., California State Dept. of Eng., on development of water supply for state institutions and survey of water resources. *TT 4: P 4: RC 2.*—June 1922 to April 1923 with City of Sacramento, until Nov. 1922 as Field Engr., on surveys for water supply and hydro-electric development, then Inspector on construction of filtration plant. *TT 0.7: SP 0.2: P 0.5: RC 0.5.*—April 1923 to Feb. 1924 Field Engr., East Bay Water Co., on survey of Eel River Project, examination and report on other sources of water for East Bay Dist. *TT 0.8: P 0.8: RC 0.8: D 0.2.*—May to Dec. 1924 Field Engr., Sacramento Municipal Utility Dist., on surveys for hydro-electric development and water supply *TT 0.7: P 0.7: RC 0.7.*—Jan. to Sept. 1925 and March 1927 to March 1928 general engineering work. *TT 1.3: SP 0.4: P 0.9.*—Oct. 1925 to May 1926 Field Engr. on reservoir surveys on American and Mokelumne River. *TT 0.7: P 0.7: RC 0.7.*—June 1926 to Feb. 1927 on land classification for assessment. *TT 0.4: SP 0.4.*—April 1928 to May 1929 Field Engr. and on design, Cosumnes River Flood Control, miscellaneous engineering. *TT 0.8: SP 0.2: P 0.6.*—June 1929 to April 1931 Hydr. Engr., Salyer Consolidated Mines Co., on design and construction, diversion works, flumes, pipe lines, reinforced concrete structures, roads, bridges and general layout for hydraulic mines. *TT 1.8: P 1.8: RC 1.8: D 0.3.*—May to Aug. 1931 Field Engr., Majestic Mines Co., Weaverville, Cal., on hydraulic mine and hydro-electric development. *TT 0.3: P 0.3.*—Sept. 1931 to date with Sacramento Municipal Utility Dist., general field and office work on water supply. *TT 0.2: P 0.2.* *TT 17.9: SP 1.1: P 16.8: RC 8.3: D 1.* Refers to P. Bailey, J. B. Brown, G. J. Calder, A. J. Cleary, S. A. Hart, S. E. Kieffer, W. S. Post, W. E. Stoddard, D. R. Warren.

1262

(11) McGINNESS, ARTHUR ROSCOE, 3554 Brayton Ave., Long Beach, Cal. (Age 47. Born Harlan, Iowa.) 1910 B. S. in Civ. and Irrigation Eng., Colo. Agri. Coll. *TT 4: P 4.*—July 1910 to March 1911 Hydrographer, Costilla Estates Development Co., San Luis, Colo., on stream flow measurement, water records and earthwork computations. *TT 0.6: SP 0.1: P 0.5: RC 0.5.*—April to June 1911 Topographer, Walther Lumber Co., Ellensberg, Wash., on topography and preliminary railroad survey. *TT 0.2: P 0.2.*—Aug. 1911 to April 1912 Rodman and Chainman, on drafting, etc., for various engineers in Ft. Collins, Colo. *TT 0.1: P 0.1.*—May 1912 to Dec. 1926 (except Jan. and Feb. 1916 Instrumentman on highway surveys in Polk County, Iowa) with U. S. Reclamation Service, as follows: May 1912 to May 1914 Canal Rider, Inspector, drainage construction, drainage investigations, Instrumentman, drafting, drain location and estimates in Savage and Huntley, Mont., and Powell, Wyo.; June 1914 to Dec. 1915 Foreman, on drainage construction work and small structures, office and field engineering, location and estimates, Huntley; March 1916 to June 1918 Supt. of Constr. on drainage system, including surveys, subsoil investigations, locations, estimates, replacement of irrigation systems, canal enlargement, construction of pumping plant, etc.; July 1918 to Aug. 1920 Asst. Engr. and Asst. Supt. of Irrigation, on drainage construction, replacement of structures, surveys and in charge of operation and maintenance work in one-half of project; Sept. 1920 to April 1921 Engr. and Supt. of Irrigation, in charge of engineering work, replacement of structure, operation and maintenance of entire project, Ballantine, Mont.; after May 1921 Engr. and Project Mgr., in full charge of activities on project. *TT 13.1: SP 1.4: P 11.7: RC 11.7: D 1.*—March 1928 to Sept. 1931 Structural Engr., City of Long Beach, Cal., on design of bridges, retaining walls, pump stations, trusses, control dams, foundations, buildings, ramps, miscellaneous structures, estimates and specifications. *TT 3.5: P 3.5: D 3.5.*—*TT 21.5: SP 1.6: P 19.9: RC 12.2: D 4.5.* Refers to A. H. Adams, C. M. Cram, R. H. Fifield, H. Paterson, A. W. Walker, F. E. Weymouth, C. P. Williams.

1263

(11) MARSH, RALPH EASTMAN, Box 1311, Tucson, Ariz. (Age 28. Born Rapid City, S. Dak.) 1927 B.S., S. Dak. State School of Mines. *TT 4: P 4.*—June to Dec. 1927 Levelman, U. S. Bureau of Public Roads, being Instrumentman on location and construction of forest highways and Inspector on bridge construction. *TT 0.4: SP 0.1: P 0.3.*—

April 1928 to date with U. S. Geological Survey, until Oct. 1931 as Jun. Engr., then Asst. Engr., on general stream gauging, 3 months in charge of Colorado River gauging stations at Topock and Yuma, Ariz., and 7 months in charge of construction of gauging stations in New Jersey and Arizona. *TT 3.6: P 3.6: RC 0.8.*—*TT 8: SP 0.1: P 7.9: RC 0.8.* Refers to D. H. Barber, W. E. Dickinson, A. Diefendorf, J. S. Gatewood, O. W. Hartwell, O. Lauterhahn, J. C. Rathbun.

1264

(1) **MARTIN, GEORGE HARRIS, Jr.,** 16 Hyatt Ave., Harrison, N. Y. (Age 33. Born Tarrytown, N. Y.) 1922 B. S. in C. E., New York Univ. *TT 4: P 4.*—July 1917 to July 1918 with Wolff Eng. Corporation, Tarrytown, N. Y., as Transitman, Chf. of Party on surveys and Inspector on road work and sewage-disposal plant. *TT 0.5: SP 0.5.*—July to Nov. 1918 with U. S. Army, R. O. T. C., Camp Lee, Va.—Jan. to Oct. 1919 and March to Oct. 1924 with County Supt. of Highways, White Plains, N. Y., as Transitman, Chf. of Party on surveys and Inspector on road and bridge construction and (after March 1924) Res. Engr. bridge construction. *TT 1: SP 0.4: P 0.6: RC 0.6.*—June 1922 to March 1924 Field Engr., Frederick Snare Corporation, syphon construction on Catskill Aqueduct. *TT 1.7: P 1.7.*—Oct. 1924 to July 1927 with Technical Dept., Atlas Lumnite Cement Co., New York City, on sales promotion work and (1 year) Eastern Technical Mgr. *TT 2.7: P 2.7.*—July 1927 to date member of firm and Mgr., Harrison Office, Chas. H. Sells, Inc., on highway design and supervision of construction, also with Survey Dept., on subdivision work, etc. *TT 4.3: P 4.3: RC 4.3: D 4.3.*—*TT 14.2: SP 0.9: P 13.3: RC 4.9: D 4.3.* Refers to M. J. Adams, A. Haring, C. R. Hulsart, J. R. Losee, C. MacDonald, C. T. Schwarze, C. H. Sells.

1265

(1) **MOHR, LAWRENCE GUSTAV,** 35 East Seventy-sixth St., New York City. (Age 22. Born Savannah, Ga.) 1930 C. E., Cornell Univ. *TT 4: P 4.*—Sept. 1930 to June 1931 graduate student, Massachusetts Inst. of Technology.—*TT 4: P 4.* Refers to K. C. Reynolds, L. C. Urquhart.

1266

(7) **MONKMAN, GEORGE HUMPHREY NELSON,** Locarno Apartments, Winnipeg, Man., Canada. (Age 43. Born Winnipeg, Man., Canada.) July 1904-Dec. 1914 with Canadian Pacific Ry., Western Lines, until July 1908 as Axeman, Chainman, Rodman, Draftsman, Instrumentman and Topographer on location and construction, then Res. Engr. in charge of grading, track work and location, during 1913 and 1914 being on bridge construction exclusively. *TT 7.8: SP 2.6: P 5.2.*—Jan. 1915 to Sept. 1919 with Canadian and British Expeditionary Forces in France and Belgium, being Capt., Royal Engrs. *TT 4.2: SP 0.5: P 3.7: RC 2: D 1.*—May 1921 to April 1923 Office Engr., Saskatchewan Prov. Federal Aid Branch, Dept. of Highways, in charge of office and organization of engineering forces. *TT 1.9: P 1.9: D 1.9.*—Sept. 1919 to Jan. 1921 and April 1923 to date with C. P. Ry., until Jan. 1921 as Res. Engr. on grading and trackwork and (6 months) Locating Engr., April 1923-Dec. 1929 Asst. Engr. in charge of all construction on various branch lines, and since Jan. 1930 Asst. Engr. of Constr., having office and field supervision of engineering parties on Western Lines. *TT 10.1; P 10.1: RC 8.8: D 0.2.*—*TT 24: SP 3.1: P 20.9: RC 10.8: D 3.1.* Refers to H. D. Brydone-Jack, J. Callaghan, F. Lee, T. C. Macnabb, J. R. C. Macredie, J. G. Sullivan.

1267

(11) **MONROE, ROY HAYDEN,** 1438 North Fremont Ave., Tucson, Ariz. (Age 28. Born Prairie Home, Mo.) 1928 B.S. in C.E., Univ. of Mo. *TT 4: P 4.*—June to Dec. 1928 Jun. Highway Engr., Illinois Highway Dept., two months in office plotting survey notes, computing grade line and earthwork and checking culvert design, then in field inspecting grading and building of concrete culverts and assisting Res. Engr. with reports. *TT 0.2: SP 0.2.*—Jan. 1929 to May 1930 Surveyman, U. S. Engrs., 2 months in field, operating level and transit and remainder of time in office, plotting and computing surveys, compiling maps and mosaics from aerial photographs, etc. *TT 0.7: SP 0.7.*—May 1930 to date Jun. Hydr. Engr., U. S. Geological Survey, office and field work of general stream gauging. *TT 1.3: SP 0.3: P 1.*—*TT 6.2: SP 1.2: P 5.* Refers to D. H. Barber, W. E. Dickinson, J. S. Gatewood, A. L. Hyde, W. A. K. Parkin, B. A. Ross, H. K. Rubey.

1268

(4) **MULLAN, ALBERT CHARLES,** 515 Sixty-seventh Ave., Philadelphia, Pa. (Age 21. Born Philadelphia, Pa.). 1931 B. S. in Civ. Eng., Pa. State Coll. *TT 4: P 4.* Refers to H. N. Benkert, H. B. Shattuck.

1269

(6) PATELLA, GIUNIO, 341 Ladson St., Pittsburgh, Pa. (Age 27. Born Piraju, Brazil.) 1926 C. E., Univ. of Naples, Italy. *TT 4: P 4.*—July 1929 to June 1930 Draftsman in Rosario Candela's Office, New York City, on building construction. *TT 0 5: SP 0.5: D 0.5.* —June to Nov. 1930 Designer in D. Briganti's Office, Bronx, N. Y., on structural steel construction. *TT 0.4: P 0.4.*—Nov. 1930 to date Transitman, Allegeny County, Pa. *TT 0.4: SP 0.4.*—*TT 5.3: SP 0.9: P 4.4: D 0.5.* Refers to L. P. Biilotta, D. J. Camilli, R. S. Quick.

1270

(4) PEARSON, WALTER HERBERT, 228 Lincoln Ave., Manoa, Pa. (Age 31. Born Plainfield, N. J.) June 1918 to May 1919 Draftsman, The Standard Roller Bearing Co., Philadelphia, Pa., roller and ball bearing application. *TT 0.4: SP 0.4.*—May 1919 to Sept. 1929 with The Baldwin Locomotive Works, Philadelphia, as Draftsman, Checker on gasoline and electric locomotives, Designer on Diesel electric locomotives, and Elevation Engr., on various types of locomotives. *TT 8.6: SP 1.8: P 6.8: RC 2.8: D 2.*—Sept. 1929 to Nov. 1930 Designing Engr., American Road Machinery Co., Kennett Square, Pa., on road-building machinery, road-oil distributors, chip spreaders, tank car-heaters, crushing and screening plants. *TT 1.2: P 1.2: D 1.2.*—Nov. 1930 to date Plant Engr., Bituminous Service Co., West Chester, Pa., in charge of plant operation and manufacture of bituminous road building equipment. *TT 1: P 1: D 1.*—*TT 11.2: SP 2.2: P 9: RC 2.8: D. 3.2.* Refers to C. C. Bailey, C. C. Campbell, R. Radbill, H. W. Underwood, S. M. Vauclain, A. S. Woodle, Jr.

1271

(13) PIERCE, ROBERT ELLIOTT, State Office Bldg., Sacramento, Cal. (Age 46. Born Oakland, Cal.) Registered Civ. Engr., State of California.—Aug. 1906 to May 1907 and Aug. 1908 to May 1909 Special Student in Civ. Eng., Univ. of California.—Jan 1901 to May 1906 with Pacific Elec. Ry., until May 1904 as Asst. Engr., then Asst. Maintenance Engr. on location and construction. *TT 2.5: SP 2.5.*—May to Aug. 1906 Locating Engr., Vallejo & Northern R. R. *TT 0.2: SP 0.2.*—May to Dec. 1907 Asst. Maintenance Engr., Northwestern Pacific R. R., on bridge construction. *TT 0.6: P 0.6: RC 0.6.*—Dec. 1907 to May 1908 Draftsman, City Gas Co., Los Angeles, Cal. *TT 0.2: SP 0.2.*—May to Aug. 1908 Asst. Maintenance Engr., Northern Elec. R. R. *TT 0.1: SP 0.1.*—May 1909 to March 1910 Asst. Engr., Huntington Land Co., Los Angeles. *TT 0.6: SP 0.3: P 0.3: RC 0.3.*—March 1910 to June 1911 Office Engr. with Dessery & West, Los Angeles. *TT 0.9: SP 0.4: P 0.5: RC 0.3: D 0.2.*—June 1911 to Feb. 1914 Asst. Engr., Dozier Cons. Co. and Haviland, Dozier & Tibbetts, Sacramento and San Francisco. *TT 1.3: SP 0.4: P 0.9: RC 0.2: D 0.7.*—Feb. 1914 to April 1915 Res. Engr., West Side R. R., Dixon, Cal. *TT 1.2: P 1.2: RC 0.6: D 0.6.*—April 1915 to date Res. Engr., Office Engr., Maintenance Engr. and (5 years) Dist. Engr., Dept. of Public Works, Div. of Highways, Sacramento. *TT 12.1: SP 4.5: P 7.6: RC 5: D 2.6.*—*TT 19.7: SP 8.6: P 11.1: RC 7: D 4.1.* Refers to C. E. Andrew, J. C. Boyd, J. Gallagher, F. J. Grumm, R. M. Morton, C. S. Pope, C. H. Purcell, T. E. Stanton, Jr.

1272

(15) PRAY, CECIL, 238 Prospect Ave., Shreveport, La. (Age 25. Born Homestead, Okla.) 1931 B. S. in Civ. Eng., Tex. Tech. Coll. *TT 4: P 4.*—June-Aug. 1931 Asst. with City of Lubbock, Tex., mapping, surveying and setting sidewalk grades. *TT 0.1: SP 0.1.*—Sept.-Oct. 1931 Cement Inspector, Texas Highway Dept. *TT 0.1: SP 0.1*—At present Asst. Eng. Aide, U. S. War Dept.—*TT 4.2: SP 0.2: P 4.* Refers to O. V. Adams, G. R. Johnston, F. L. McRee, J. H. Murdough.

1273

(1) REA, FLORENTINO GUERINO LUIS, 455 Boulevard Avellaneda, Rosario, Sante Fe, Argentine Republic. (Age 26. Born Rosario de Santa Fe, Argentina.) 2 years student, Escuela Industrial de la Nacion, Facultad de Ciencias Escactas y Naturales del Rosario. —April to June 1928 Draftsman and Computer with F. French Paddock & Associates, Cons. Engrs., Detroit, Mich., on land development. etc. *TT 0.2: P 0.2: RC 0.2: D 0.2.* —Dec. 1928 to Dec. 1929 Chf. Draftsman, Eng. Dept., Roseville Village, Mich., planning and designing water and lateral sewer systems. *TT 1: P 1: RC 1: D 1.*—Jan. to Aug. 1930 Engr. with H. R. O'Mara, C.E., East Detroit, Mich., on design and layout of land subdivision, water and sewer systems, etc. *TT 0.7: P 0.7: RC 0.7: D 0.7.*—Sept. 1930 to Jan. 1931 with Wm. Benton Constr. Co., Mt. Clemens, Mich., in charge of field engineering on construction of government officers' quarters (32) at Selfridge Aviation Field. *TT 0.4: P 0.4: RC 0.4.*—Dec. 1925 to March 1928 Draftsman, Computer, Designer and Field Engr. on construction of reinforced concrete and asphaltic roads,

July to Nov. 1928 Asst. Bridge Engr. designing reinforced concrete bridges, and Feb. 1931 to date Asst. to Field Engr. on construction of reinforced concrete roads, all for Macomb County (Mich.) Road Comm.; also (spare time) on design, plat and field layout of Lincoln Memorial Park Cemetery (33 acres) in Macomb County for Macomb County Road Comm. (March 1928); with Greater Gratiot Promotion Comm. on plan studies, perspective works, bird's eye view of proposed highway through Mt. Clemens (June 1929); design of land subdivision for Dopp Bros. of Mt. Clemens, including field layout, descriptions, etc. (March 1931); 4 years (prior to Aug. 1926) with Frezza, Pinchetti y Cia, Rosario, as Designer of small shops, shop extensions, layouts, etc. *TT 2.5: SP 1: P 1.5: RC 1.5: D 0.7.*—*TT 4.8: SP 1: P 3.8: RC 3.8: D 2.6.* Refers to W. H. Dorrance, 4th, W. C. Dotson, W. J. Lehner, D. G. Ramsay, M. Van de Greyn.

1274

(2) **ROEMER, EDWARD WILLIAM,** 901 City Hall Annex, Boston, Mass. (Age 50. Born Brookline, Mass.) Nov. 1900 to June 1905 Apprentice, Mason and Foreman with Woodbury & Leighton, Boston, Mass., estimating, laying out work and in charge of building construction.—June to Sept. 1905 Mason with George A. Fuller Co., Boston, and with other building contractors.—Sept. 1905 to May 1906 Foreman Mason, L. P. Soule Co., in charge of crew of masons on construction of cold storage plant in Boston.—Aug. 1906 to Oct. 1907 Supt. of Constr., until Feb. 1907 with H. P. Converse Co., Boston, on erection of office building in Brandford, Conn. then with A. B. Murdough, Watertown, constructing group of hospital buildings for Commonwealth of Massachusetts in Canton, Mass. *TT 1.1: P 1.1: RC 1.1.*—1907 to 1910 Contr., erecting small buildings, subcontracting for foundations and other masonry work and jobbing. *TT 3: P 3: RC 3.*—March 1910 to date with City of Boston, until April 1911 as Clerk of Works, Schoolhouse Dept., inspecting construction of school buildings, then with Bldg. Dept., 2 years as Bldg. Inspector, 3 years Chf. of Application Desk, examining plans and applications for building permits, giving information relative to building law and building construction, April 1916 to May 1930 Supervisor of Constr., being Asst. to Bldg. Commr. and in direct charge of inspectors on construction; consulted with architects, engineers and building contractors on materials, methods of construction and design, and since May 1930 Bldg. Commr., in charge of administration and enforcing of Building, Zoning, Plumbing, and Gasfitting Laws, State Elevator Regulations, etc., pertaining to erection and maintenance of buildings in Boston. *TT 21.7: P 21.7: RC 14.5: D 7.2.*—*TT 25.8: P 25.8: RC 18.6: D 7.2.* Refers to B. S. Brown, J. E. Carty, L. A. Estes, M. Linenthal, L. K. Rourke, T F. Sullivan.

1275

(1) **SAMUELS, ABRAHAM SOLOMON,** 2710 Morris Ave., New York City. (Age 24. Born New York City.) Sept. 1926 to Jan. 1927 (evenings) student, Columbia Univ.—April 1924 to Sept. 1925 Supt. and Estimator with Philip Samuels, New York City, laid out and supervised work. *TT 0.7: SP 0.7.*—Sept. 1925 to Dec. 1926 Asst. Supt. and Estimator, Tessler & Albert, Inc., Bldrs. and Plumbing Contrs., New York City, checked plumbing and heating work, gave line and grades for buildings. *TT 0.7: SP 0.7.*—Dec. 1927 to June 1929 in private practice, roofing and contracting, being Owner, Supt. and Estimator *TT 1.2: SP 0.2: P 1: RC 1.*—Sept. to Dec. 1927 and June 1929 to date Engr. Asst., Board of Transportation, New York City, being Instrumentman and (since June 1929) Asst. Chf. of Party, giving lines and grades for excavation, steel, grades for concrete and street pavements. *TT 2.7: SP 0.2: P 2. 5.*—*TT 5.3: SP 1.8: P 3.5: RC 1.* Refers to H. J. Alexander, F. J. Easterbrook, J. H. Myers, R. Ridgway, C. D. Thomas.

1276

(11) **SEDGWICK, ALLAN ERNEST,** 903 South Highland Ave., Los Angeles, Cal. (Age 50. Born York, Nebr.) Licensed Archt. and Civ. Engr., State of California.—1918 B. S. in C. E. and 1919 M. S., Univ. of So. Cal.—1902 to 1906 Student, Columbia Univ.—1905 to 1906 Engr., Greenback Gold Min. Co., in charge of cyanide plant and building of aerial tramway. *TT 0.7 P 0.7: RC 0.7.*—1907 to 1908 Engr., Tezuitlan Copper Co., being Asst. Supt. of Mines, in charge of mining and Engr. of Constr. on hydro-electric plant. *TT 1.5: P 1.5: RC 1.5.*—1908 to 1910 Gen. Mgr., American Eng. & Constr. Co., Mexico, D. F., general contracting and railroad construction. *TT 1.4: P 1.4: RC 1.4.*—1910 to 1912 Gen. Mgr., Maine & Nebraska Min. Co., mining and smelting copper ore. *TT 2.2: P 2.2: RC 2.2.*—1913 to 1916 Designing Engr., Noonan & Richards, Archts., Los Angeles, Cal., on design of reinforced concrete and steel construction, testing materials and superintending. *TT 2.2: P 2.2: D 2.2.*—1916 to 1917 Engr., Avawatz Salt & Gypsum Co. *TT 1: P 1: RC 1.*—1917 to 1918 Engr., Dominguez Land Corporation, subdivisions, street and drain-

age work. *TT 1: P 1: RC 1.*—1918 to 1929 with Univ. of Southern California, 2 years as Asst. Prof., 3 years Associate Prof., and 3 years Prof. of Geology and Head of Dept., and 3 years Prof. of Geology and Head of Dept. of Petroleum Eng. *TT 11: P 11. RC 11.* —1929 to date in private practice as Cons. Engr., Los Angeles, Cal., being Member of Comm. of Los Angeles County to investigate failure of St. Francis Dam (1928) and of Comm. of Los Angeles County Flood Control to investigate San Gabriel Dam site (1929) and on examination of Tujunga Dam site (1930); Cons. Geologist, with Dept. of Water & Power, City of Los Angeles (1928–1929) and city of Santa Barbara (1929–1930), with Montecito County Water Dist., County of Santa Barbara (1929), with Merritt, Chapman, Scott Co., quarry operations (1929 to 1931) and with Golden Gate Bridge & Highway Dist., San Francisco, (1931). *TT 2.5: P 2.5.*—*TT 23.5: P 23.5: RC 18.8: D 2.2.* Refers to L. C. Hill, P. E. Jeffers, C. T. Leeds, J. B. Lippincott, F. A. Noetzli, H. Z. Osborne.

1277

(2) **SKELTON, RUSSELL ROY,** Madbury Road, Durham, N. H. (Age 35. Born Princeton, Ind.) 1923 B. S. in C. E., Purdue Univ. *TT 4: P 4.*—1918 to 1919 with Air Service, U. S. Army, 3 months in Ground School, Univ. of Illinois and 10 months at Ellington Field, Houston, Tex.; commissioned 2d Lieut.—June 1923-March 1926 Jun. Engr., Illinois State Highway Dept., Peoria, Ill., being Instrumentman, Chf. of Survey Party, Asst. Res. Engr. and Res. Engr., on trunk-line construction, design of concrete roads, grade line and special intersections. *TT 2.8: P 2.8: RC 2: D 1.5.*—April 1926-Aug. 1928 Jun. Engr. and Asst. Engr., Southern Ry. System, Bridge Office, Maintenance of Way and Structures, Cincinnati, Ohio, on bridge surveys, inspection and reports, design of concrete, steel and timber structures, and investigations for grade-crossing elimination. *TT 2.4: P 2.4: RC 2.4: D 2.4.*—Sept. 1928 to Sept. 1930 Instructor in, and Sept. 1930 to date Asst. Prof. of, Civ. Eng., Univ. of New Hampshire. *TT 3.2: P 3.2: RC 1.2.*—*TT 12.4: P 12.4: RC 5.6: D 3.9.* Refers to E. W. Bowler, G. W. Case. F. E. Everett, W. A. Grover, W. K. Hatt, W. A. Knapp, G. E. Martin, R. B. Wiley.

1278

(12) **SMITH, MARVIN WENDELL,** 419 Commerce St., Everett, Wash. (Age 51. Born Glen Allen, Mo.) 1 year student, Univ. of Washington.—1903 to 1910 in charge of platting and checking survey, abstracting and conveying titles, some instrument work, etc., City of Mulkito, Wash., for Mulkito Land Co., and similar work for subdivision of Darlington for Dwight Darling. *TT 5: SP 2: P 3.*—1910 to 1913 in charge of platting 1 300-acre subdivision, San Louis Obispo County, Cal. *TT 3: P 3: RC 3: D 1.*—1913 to 1920 handling sales and platting Glenwood Subdivisions A and B, Snohomish County, Wash. *TT 7: P 7: RC 7.*—1920 to date Secy., Everett (Wash.) Port Comm., on construction and design of port wharf, Mukilto (Everett Port Dist.) (1926), in charge of design of plan for port development (1925), of design and construction of small-boat haven and industrial site, Everett Port (1927–1928) and of city airport, Everett (1929), and in charge of port development, including channel dredging, revetment work and filling industrial sites (1930). *TT 11: P 11: RC 11: D 2.*—*TT 26: SP 2: P 24: RC 21: D 3.* Refers to J. C. Greely, S. H. Hedges, H. W. McCurdy, H. Mumm, Jr., O. A. Piper.

1279

(14) **SNYDER, LAWRENCE KING,** Rolla, Mo. (Age 25. Born Linn Creek, Mo.) 1929 B. S. in Civ. Eng., Mo. School of Mines. *TT 4: P 4.*—Summer 1928 and July to Sept. 1929 with Missouri State Highway Dept., Sikeston, Mo., as Rodman, Inspector at proportioning plant on concrete paving project, and (1929) Draftsman. *TT 0.1: P 0.1.*—Feb. to July 1929 Designer, Bridge Dept., Cleveland, Cincinnati, Chicago & St. Louis Ry. Co., Cincinnati, Ohio, detailing concrete and steel structures. *TT 0.5: P 0.5.*—Sept. 1929 to Feb. 1931 Bridge Draftsman, St. Louis-San Francisco Ry. Co., St. Louis, Mo. *TT 1.4: P 1.4.*—*TT 6: P 6.* Refers to C. E. Bardsley, J. B. Butler, E. W. Carlton, E. G. Harris, F. G. Jonah, C. V. Mann.

1280

(15) **STAPLES, LEROY AUGUSTUS,** 5101 Pitt St., New Orleans, La. (Age 27. Born Houma, La.) 1926 B. S. in M. E., Ga. School Tech. *TT 4 P 4.*—July 1926 to Oct. 1930 (except March to Aug. 1930) with Jones & Laughlin Steel Corporation, designing, detailing and checking structural steel for buildings and bridges. *TT 2.5: SP 1.3: P 1.2: RC 1.2: D 1.2.*—March to Aug. 1930 and Oct. 1930 to date with George P. Rice, Cons. Engr., New Orleans, La., on design of reinforced concrete and steel structures. *TT 1.6: P 1.6: RC 1.6: D 1.6.*—*TT 8.1: SP 1.3: P 6.8: RC 2.8: D 2.8.* Refers to J. H. Cherry, P. D. Cook, J. de Tarnowsky, P. C. Kuhn, J. A. Miller, R. L. Moroney, G. P. Rice.

1281

(3) STAUNING, GEORGE, Box 42, Rosendale, N. Y. (Age 37. Born New York City.) Licensed Prof. Engr. and Land Surveyor, State of New York.—Student, Tri State Coll. and (night school) Columbia Univ., City Coll. of N. Y. and Mechanics Inst. (1 term at each).—1914 to 1919 Rodman to Chf. of Party, City of Stamford, Conn., general municipal engineering, sidewalks, parks, pavements, sewers, etc. *TT 4: SP 1: P 3: RC 3: D 3.* —1919 to 1922 Instrumentman, New York Central R. R., and 1922 to 1923 Engr., Union Pacific R. R., Oregon Short Line, on track, yard and building layout, design, estimates and construction. *TT 4: P 4: RC 4: D 4.*—1923 to 1924 Engr., City of Akron, Ohio, on estimates, design, construction, sidewalks and pavements. *TT 1: P 1: RC 1: D 1.*—1924 to 1929 Maintenance Engr., North American Cement Corporation, Catskill, N. Y., on plant design and construction, in charge of quarry and plant operations. *TT 5: P 5: RC 5: D 5.* —1929 to 1930 Quarry Supt., Merritt-Chapman & Scott Corporation, New York City, in charge of land operations, breakwater construction, Manistique, Mich. *TT 1: P 1: RC 1.*— April 1930 to date Gen. Supt., Century Cement Corporation, Rosendale, N. Y., in charge of all cement plant operations. *TT 1.6: P 1.6: RC 1.6: D 1.6.*—*TT 16.6: SP 1: P 15.6: RC 15.6: D 14.6.* Refers to R. W. Atwater, A. Diefendorf, E. A. Kemmler, J. F. Loughran, P. Nash.

1282

(5) THOMSON, MEDFORD THEODORE, 1724 Q St. N. W., Washington, D. C. (Age 27. Born Buffalo, N. Y.) 1925 C. E., Cornell Univ. *TT 4: P 4.*—June 1925 to May 1926 Rodman, Pennsylvania R. R. System, on drafting, surveying and maintenance of way. *TT 0.9: P 0.9.*—June 1926 to date with Water Resources Branch, U. S. Geological Survey, until Dec. 1928 as Jun. Engr., then Asst. Engr., collecting and compiling basic data for stream-flow records and on construction of equipment for gauging stations. *TT 5.5: P 5.5: RC 4.5.*—*TT 10.4: P 10.4: RC 4.5.* Refers to F. A. Barnes, N. C. Grover, H. F. Hill, Jr., A. H. Horton, J. C. Hoyt, W. Kessler, W. R. King.

1283

(8) WALKER, JAMES DONALD, 705 Pennsylvania Ave., Aurora, Ill. (Age 24. Born Decatur, Ill.) 1930 B. S., Univ. of Ill. *TT 4: P 4.*—June 1930 to date San. Engr., The American Well Works, Aurora, Ill., investigating and developing new type of bio-aeration apparatus for sewage treatment. *TT 1.4: P 1.4: RC 1.4.*—*TT 5.4: P 5.4: RC 1.4.* Refers to W. D. Gerber, W. C. Huntington.

1284

(4) WEHMEYER, LOUIS FRANKLIN, 7348 Palmetto St, Philadelphia, Pa. (Age 59. Born Philadelphia, Pa.) 1886 to 1889 student in mechanical engineering, Spring Garden Inst., Philadelphia.—1889 to 1899 Machinist, Draftsman and Erector for various firms. *TT 8 SP 2: P 6.*—1899 to 1900 with Tacony Iron & Metal Co., Philadelphia, on design and detailing structural iron work, bridges, etc. *TT 1: P 1: D 1.*—1900 to 1907 with Reading Railroad Co., in Chf. Engr.'s office on design of fixed and movable bridges and masonry structures. *TT 7: P 7: D 7.*—1907 to date with Bureau of Eng. and Surveys, City of Philadelphia, being Chf. Draftsman of Bridge Div., on design of movable and fixed bridges and masonry structures. *TT 24: P 24: RC 24: D 24.*—*TT 40: SP 2: P 38: RC 24: D 32.* Refers to C. Dillenbeck, J. Jones, J. H. Neeson, S. H. Noyes, H. H. Quimby.

1285

(9) WEIGAND, EDWIN JOHN, 400 Marion E. Taylor Bldg., Louisville, Ky. (Age 35. Born Cincinnati, Ohio.) 1918 B.S. in C.E., Ohio State Univ. *TT 4: P 4.*—July 1918 to Feb. 1919 Ensign, U. S. Navy, training at U. S. Navy Steam Eng. School at Stevens Inst. of Technology; Jun. Watch Engr. aboard *U. S. S. Kittery.* *TT 0.7: P 0.7.*—March to Oct. 1919 Instrumentman with C. D. Putnam, Civ. and Landscape Engr., Dayton, Ohio, on surveys. *TT 0.3: SP 0.3.*—Nov. 1919 to Feb. 1923 Asst. Engr., Miami Conservancy Dist., Dayton, Ohio, on design and hydraulic study of flood-control structures and design of concrete girder and arch bridges, retaining walls and steel bridges. *TT 3.3: P 3.3: D 3.3.*—March 1923 to date with Commrs. of Sewerage of Louisville, Ky., until Dec. 1924 Jun. Engr., on hydraulic and structural designs of sewerage systems ($2 000 000 sewer program), and since Jan. 1925 Senior Engr., in responsible charge of Design and Drafting Dept., supervising preliminary investigations for plans of sewerage for City of Louisville, and later supervising hydraulic and structural designs and contract plans and specifications for sewerage systems of $15 000 000 sewer program. *TT 8.7: P 8.7: RC 6.9: D 8.7.*—*TT 17: SP 0.3: P 16.7: RC 6.9: D 12.* Refers to W. M. Caye, J. M. Johnson, J. H. Kimball, C. H. Paul, C. D. Putnam, R. M. Riegel, C. E. Sherman.

1286

(16) WELDEN, RICHARD WAYNE, 710 Union St., Iowa Falls, Iowa. (Age 23. Born Iowa Falls, Iowa) 1931 B. S. in C. E., Iowa State Coll. *TT 4: P 4.*—June 1931 to date Supt. of Constr., Welden Bros., Iowa Falls, Iowa, in charge of gunite and concrete construction. *TT 0.3: P 0.3: RC 0.3.* *TT 4.3: P 4.3: RC 0.3.* Refers to R. A. Caughey, J. R. Maher, Sr., E. Welden.

1287

(2) WILBUR, JOHN BENSON, 410 Memorial Drive, Cambridge, Mass. (Age 27. Born Oakland, Me.) 1926 B. S., and 1928 M. S., Mass. Inst. Tech. *TT 4: P 4.* Summer 1925 Inspector, Portland Cement Association, Kenmore, N. Y., on concrete highway construction. —June 1926 to June 1928 Asst. in Civ. Eng., Massachusetts Inst. of Technology; summers Asst. at Surveying Camp; Nov. 1927 to March 1928 (spare time) also with Fay, Spofford & Thorndike, Boston, Mass., on design of Lake Champlain Bridge, made exact solutions for continuous trusses and preliminary design of suspension bridge. *TT 2: P 2.*— June 1928 to Aug. 1929 Structural Draftsman, Bridge Dept., Maine Central R. R., on design of masonry and superstructure, including falsework and erection, investigation of existing bridges, fixing truck capacity for all overhead bridges. *TT 1.2: P 1.2: RC 1.2: D 0.7.*—Aug. 1929 to Sept. 1930 and June to Sept. 1931 Designer and Detailer, Bridge Dept., New York Central R. R., drafting, designing and checking on bridge over Bronx River Parkway Drive at Woodlawn, N. Y., 241st St. Viaduct at Wakefield, N. Y., elevated express highway and West Side Improvement, New York City, also checking design plans for latter. *TT 1.5: P 1.5: RC 0.3: D 1.*— Sept. 1930 to June 1931 and Sept. 1931 to date Instructor in Civ. Eng., Massachusetts Inst. of Technology, teaching bridge design and hydraulics and advanced structural design, being in charge of course since Jan. 1931. *TT 0.9: P 0.9: RC 0.9.*—*TT 9.6: P 9.6: RC 2.4: D 1.7.* Refers to J. B. Babcock, 3d, W. M. Fife, W. H. Norris, C. M. Spofford, C. H. Sutherland, H. T. Welty.

1288

(15) WOODYARD, FRANK ALBERT, Apartado 397, Monterrey, Mexico. (Age 33. Born Parkersburg, W. Va.) 1917 to 1920 student, Texas School of Mines (2 years) and Univ. of Southern California (1 year, nights). *TT 1: P 1.*— Oct. to Nov. 1918 at Infantry Officers' Training School, Camp MacArthur, Waco, Tex.—1919 to 1920 with Myron Hunt, Archt., Los Angeles, Cal., as Inspector on Los Angeles Ambassador Hotel. *TT 0.6: SP 0.6.*—1920 to 1921 with MacDonald & Kahn, Gen. Contrs., Los Angeles, as Timekeeper, Materialman, and Transitman, laying out buildings. *TT 0.4: SP 0.4.*—1921 to 1922 Supt. of Constr. for Swasey & McAfee, at Lake Arrowhead, Cal., in responsible charge of construction of two hotel and five store buildings, a clubhouse, a dance pavillion, boathouses and twenty-five residences ($10 000 to $50 000 each.) *TT 1.6: P 1.6: RC 1.6.* —1922 to 1923 Gen. Supt. of Constr., Norris-Schilz Co., Gen Contrs., Los Angeles and Pasadena, having complete charge of building schools, warehouses, residences and store buildings. *TT 0.6: P 0.6: RC 0.6.*—1927 to 1928 not on engineering work—1923 to 1927 with J. F. Woodyard, Jr., and 1928 to date member of firm, J. F. Woodyard, Jr., & Son, Monterrey, Mexico, directing construction of hotels, residences, warehouses, store and office buildings, flour mills, bulk stations for oil companies, cement plant, gasoline stations and light and power plants. *TT 7: P 7: RC 7.*—*TT 11.2: SP 1: P 10.2: RC 9.2.* Refers to J. H. Brillhart, H. S. Crocker, J M. Howe, T. E. Huffman, D. Lee, H. C. Poske, C. L. Tindall, J. F. Woodyard, Jr.

FOR TRANSFER

FROM THE GRADE OF ASSOCIATE MEMBER

1289

(7) CARTER, HAROLD SAMUEL, Assoc. M., 807 Eighth St., Brookings, S. Dak. (Elected Junior Jan. 14, 1924; Assoc. M. March 14, 1927.) (Age 35. Born North Evans, N. Y.) 1921 B. S. in C. E., Ore. State Agri. Coll. 1923 M. S. in C. E., and 1929 C. E., Iowa State Coll. *TT 4: P 4*—College vacations, Jan. to Sept. 1919 and summer 1921 Chairman, Rodman, Instrumentman, Draftsman and Inspector, Oregon Highway Comm., on location, grading, pavements and small bridges. *TT 0.7: P 0.7: RC 0.7.*—June 1918 to Jan. 1919 at Reserve Officers' Training Camp and with 213th F. S. Bn., Co. C, U. S. Army— June 1921 to June 1923 Graduate Asst. and Graduate Student, and Sept. 1923 to June 1924 Instructor, C. E. Dept., Iowa State Coll.; summers 1922 and 1923 Asst. Research Engr., Bureau of Public Roads, Ames, Iowa, on highway research. *TT 1.3: P 1.3: RC 0.5.*—June to July 1924 Inspector, Currie Eng. Co., Webster City, Iowa, on sheet asphalt repair work at Ft. Dodge, Iowa. *TT 0.1: P 0.1: RC 0.1.*—Sept. 1924 to date with Civ.

Eng. Dept., South Dakota State Coll., 1 year as Asst. Prof., 1 year Associate Prof. and Acting Head of Dept., and since Sept. 1926 Prof. and Head of Dept.; summers 1926 and 1927 Instructor in Highway Eng., and Research Asst., Iowa State Coll. since Sept. 1923 also on consulting work and Designer on reinforced concrete heating tunnels, water reservoir, low concrete dam, city water-works, etc., and surveying and mapping ($150 000) and Engr. in charge of college building construction ($160 000). *TT 7.2: P 7.2: RC 7.2: D 7.2.*—*TT 13.2: P 13.2: RC 8.5: D 7.2.* Refers to T. R. Agg, H. B. Blodgett, J. M Brown, J. S. Dodds, W. L. Foster, A. H. Fuller, H. S. Rogers.

1290

(8) **CHILDS, JAMES HENDERSON**, Assoc. M., Y. M. C. A. Bldg., Sterling, Ill. (Elected Nov. 21, 1921.) (Age 46. Born Forsyth, Ga.) 1904 B. S. in C. E., Ala. Pol. Inst. 1908 Ph.B., Sheffield Sci. School. *TT 4: P 4.*—July 1908 to July 1909 and Aug. 1911 to Jan. 1912 with U. S. Reclamation Service as Surveyman, etc. *TT 0.8: SP 0.5: P 0.3.*—Aug. 1909 to Dec. 1910 Draftsman, until Dec. 1909 with American Bridge Co., Chicago, then with Sacramento Valley Irrigation Co. *TT 1.2: SP 0.2: P 1.*—March to Aug. 1911 with Los Angeles (Cal.) Aqueduct, on topographic surveys. *TT 0.4: P 0.4.*—Jan. 1912 to July 1913 Surveyor, 1 year with Salt River Valley Project, Mesa, Ariz., then with Imperial (Cal.) Irrigation Dist., Water Co. No. 1. *TT 1.5: P 1.5.*—July 1913 to July 1914 with Pacific Elec. R. R. Co., Los Angeles, on valuation. *TT 1: P 1.*—July 1914 to July 1917 Deputy County Surveyor, Los Angeles County. *TT 3: P 3: RC 3.*—July 1917 to May 1919 1st Lieut. Engrs., U. S. Army. *TT 0.9: SP 0.9.* May-Dec. 1919 Chf. of Field Surveys, Los Angeles Flood Control. *TT 0.5: P 0.5: RC 0.5.*—Jan. to April 1920 Asst. Engr., Southern California Edison Co., Big Creek, Cal. *TT 0.2: P 0.2.*—April to May 1920 Draftsman, Foundation Co. *TT 0.2: P 0.2: D 0.2.*—June 1920 to June 1921 Asst. Engr., J. G. White & Co. *TT 1: P 1: RC 1.*—June 1921 to July 1922 Asst. Engr., Water Dept., City of Pasadena. *TT 1: P 1: RC 1.*—July 1922 to May 1923 Asst. Engr., U. S. Indian Service, Blackfoot, Idaho. *TT 0.7: P 0.7: RC 0.7.*—May to Nov. 1923 Engr., Industrial Terminal R. R. Co. (Southern Pacific R. R.), Los Angeles. *TT 0.6: P 0.6: D 0.6.* Nov. 1923 to Nov. 1924 San. Engr., Health Dept., City of Los Angeles. *TT 1: P 1: RC 1.*—Jan. 1925 to July 1930 Cons. Engr., Los Angeles, made report on sewerage problems for various cities and municipalities (1925), made survey and report and acted as expert witness on canal system of Carl Pleasant Dam and Ramola Grapefruit Project (1926), supervised construction of schools, apartment houses, club buildings and roads for Metropolitan Casualty Insurance Co. (1927–1928) and checked design of public buildings and acted as expert witness (1929–1930). *TT 5.2: SP 0.2: P 5: RC 5: D 1.*—July 1930 to date Associate Engr., U. S. Engrs. Office, Rock Island, Ill. *TT 1.3: P 1.3: RC 1.3: D 0.5.*—*TT 24.6: SP 1.9: P 22.7: RC 13.5: D 2.3.* Refers to J. H. Dockweiler, G. E. Edgerton, J. J. Jessup, H. Z. Osborne, A. M. Rawn, D. L. Reaburn, S. Storrow.

1291

(2) **COX, WILLIAM JUNKIN**, Assoc. M., 51 Prospect St., New Haven, Conn. (Elected Junior, Sept. 9, 1919; Assoc. M., Dec. 5, 1927.) (Age 35. Born Portland, Ore.) 1917 B. A., 1918 B. S. in C. E., and 1928 C. E., Washington and Lee Univ. *TT 4: P 4.*—Summer 1915 Field Asst., Virginia Geological Survey.—Summer 1916 on topographic survey, and Sept. 1917 to May 1918 (while student) Instructor in Civ. Eng., Washington and Lee Univ.—June 1918 to Aug. 1919 2nd Lieut., 605th Engrs., U. S. Army (A. E. F.). *TT 1.2: P 1.2.*—Sept. to Dec. 1919 Rodman and Instrumentman with H. E. Fox, Civ. and Min. Engr., Big Stone Gap, Va. *TT 0.1: SP 0.1.*—Jan. to June 1920 Instructor, Dept. of Eng. Mechanics, Sheffield Scientific School, Yale Univ. *TT 0.5: P 0.5.*—June 1920 to April 1922 Draftsman in field office of Chas. T. Main on construction of refinery for American Sugar Refining Co., Baltimore, Md. *TT 1.2: SP 0.7: P 0.5.*—April to Sept. 1922 with Spencer Constr. Co., Baltimore, drafting, detailing, estimating and some minor design on mill buildings and concrete bins. *TT 0.5: P 0.5.*—Sept. 1922 to Jan. 1923 Asst. Engr. and Chf. of Field Parties, J. A. P. Crisfield Contr. Co., on preliminary topographic surveys for hydro-electric project on Housatonic River, Conn. *TT 0.3: P 0.3: RC 0.3.*—Jan. 1923 to Oct. 1924 (part of time ill) Traffic Engr., National Bureau of Casualty & Surety Underwriters, New York City, making studies of causes and means for prevention of automobile accidents. *TT 1: P 1: RC 1: D 0.5.*—Oct. 1924 to Dec. 1926 ill: about 2 months research, etc., on automobile hazards in cities. *TT 0.2: P 0.2: RC 0.2.*—Jan. to June 1927 Instructor, Dept. of Eng. Mechanics, and July 1927 to date Asst. Prof. of Eng. Mechanics, Sheffield Scientific School, Yale Univ. *TT 4.9: P 4.9: RC 4.4.*—*TT 13.9: SP 0.8: P 13.1: RC 5.9: D 0.5.* Refers to A. B. Barber, C. T. Bishop, R. S. Kirby, P. G. Laurson, W. T. Lyle, H. B. Rust, C. J. Tilden, J. C. Tracy, E. W. Wiggin.

1292

(1) FINEBAUM, HARRY JACOB, Assoc. M., 41 West Ninety-sixth St., New York City. (Elected Nov. 25, 1919.) (Age 38. Born London, England.) 1912 B. E. Cooper Union Inst. Tech. 1914 C. E., Pol. Inst. of Brooklyn. *TT 4: P 4.*—Nov. 1912 to April 1917 Draftsman-Designer, 6 months with Otis Elevator Co., New York City, on design of steelwork for elevators, then with Eng. Dept., New York Municipal Ry. Corporation, Brooklyn, N. Y., on design and plans of elevated railroad system of Brooklyn Rapid Transit Co. *TT 3.7: SP 0.5: P 3.2: D 3.7.*—April 1917 to June 1918 and May 1920 to Nov. 1924 with Clarence W. Hudson, New York City, on Hill to Hill Bridge (6 200 ft., reinforced concrete), until June 1918 as Asst. Engr. on survey and design, May 1920 to July 1921 Prin. Asst. Engr. in charge of office during design and planning for revised design, and after July 1921 Res. Engr. in charge of supervision of construction of the bridge, including design of all revisions in field. *TT 5.7: P 5.7: RC 5.7: D 5.7.*—June to Dec. 1918 with U. S. Army, after Aug. 1918 at Engr. Officers' Training School, Camp Humphreys, Va.—Jan. 1919 to April 1920 Engr., Shipyard Plants Div., Emergency Fleet Corporation, Philadelphia, Pa. on design and plans for floating dry docks. *TT 1.3: P 1.3: RC 1.3: D 1.3*—Nov. 1924 to Jan. 1927 Chf. Engr., and Jan. 1927 to July 1929 Vice-Pres. and Constr. Mgr., Potruch Constr. Co., Allentown, Pa., in charge of engineering and construction, involving housing developments, apartment houses, theatre, factory buildings, hotels, etc. *TT 4.7: P 4.7: RC 4.7: D 4.7.*—Aug. 1929 to Nov. 1931 Project Mgr. and Engr., Gresham Constr. Co., New York City, on construction and development of large apartment houses and office buildings, having particular charge of several of these operations. *TT 2.2: P 2.2: RC 2.2.*—Nov. 1931 to date in private practice.—*TT 21.6: SP 0.5: P 21.1: RC 13.9: D 15.4.* Refers to G. H. Blakeley, F. O. Dufour, R. J. Fogg, C. W. Hudson, C Mayer, S. Smith, F. H. Snow, L. White, F. P. Witmer.

1293

(9) POWELL, RALPH WATERBURY, Assoc. M., 75 West Norwich Ave., Columbus, Ohio. (Elected July 16, 1928.) (Age 42. Born Ionia, Mich.) 1911 B.S., Mich. State Coll. 1914 C.E., Cornell Univ. 1916 Ph.B., Sheffield Sci. School, Yale Univ. *TT 4: P 4.*—Sept. 1911 to Aug. 1914 Instructor in Civ. Eng., Michigan State Coll. and (after Sept. 1912) Cornell Univ. *TT 3: P 3.*—Sept. 1914 to Aug. 1916 Halftime Asst. in Testing Materials, Sheffield Scientic School, teaching mechanics and strength of materials. *TT 1: P 1.*—Sept. 1916 to Aug. 1922 Head of Physics Dept., and July 1923 to April 1927 Associate Prof. in charge of Dept. of Applied Mechanics, College of Yale-in-China, Changsha, China; also Coll. Surveyor and (at times) in charge of building operations at Yale-in-China (1918 to 1927); Chf. Engr., Paotingfu Project, American Red Cross, China Famine Relief (May–July 1921); Member of Board and later Director of Maintenance of Siangtan-Paoching Rd., Hunan, China (Jan. 1922 to Jan. 1927). *TT 4.3: P 4.3: RC 1.4: D 0.2.*—Oct. to Dec. 1922 Jun. Engr., in office of City Engr., New Haven, Conn., being Instrumentman, Draftsman and Computer. *TT 0.1: SP 0.1.*—Jan. to May 1923 Jun. Engr., in office of Technical Advisory Corporation of New York on zoning ordinance for New Haven, being Map Draftsman. *TT 0.2: SP 0.2.*—May to Sept. 1927 Jun. Engr., U. S. Engr. Office, Chattanooga, Tenn., designing (under direction) Cove Creek Dam Project, flood probability and storage regulation studies. *TT 0.3: P 0.3.*—Sept. 1927 to date Asst. Prof. of Mechanics, Ohio State Univ.; also July to Aug. 1928 Asst. Engr., Orange County Flood Control Dist., and June to Sept. 1930 Asst. Engr., U. S. Engr. Office, Rock Island, Ill., on investigation of possibility of maintaining navigable depth in upper Mississippi by reservoirs on tributaries. *TT 4.2: P 4.2: RC 4.2.*—*TT 17.1: SP 0.3: P 16.8: RC 5.6: D 0.2.* Refers to E. F. Coddington, C. D. Curtiss, E. W. Lane, F. A. Nagler, P. W. Ott, C. E. Sherman, W. B. Wendt.

1294

(15) REAM, WILBUR BARNER, Assoc. M., 1205 Las Lomas Rd., Albuquerque, N. Mex. (Elected Oct. 14, 1929) (Age 36. Born Lewisburg, Pa.) 1920 B. S., and 1927 C. E., Bucknell Univ. *TT 4: P 4.*—June 1920 to April 1925 with C. B. Drake, Cons. Engr., Harrisburg, Ill., inspecting, chaining and drafting, Chf. of Party, superintending party chiefs, design of sewers, bridges and coal mine layouts. *TT 3.8: SP 1: P 2.8:*—June 1925 to June 1926 Asst. Engr. and Secy., Bridgeman & Allen, Inc., Cons. Engrs., in charge of office involving design and direction of field parties. *TT 1: P 1: RC 1: D 1.*—June to Aug. 1926 special design work, Croissantania Development Corporation. *TT 0.2: P 0.2: RC 0.2: D 0.2.*—Oct. 1926 to date with Middle Rio Grande Conservancy Dist., drafting, checking and computing, being Asst. in designing and Designing Engr. *TT 5.1: P 5.1:*

RC 3.6: D 3.6.—*TT 14.1: SP 1: P 13.1: RC 4.8: D 4.8.* Refers to J. L. Burkholder, C. C. Chambers, R. L. Cooper, A. Diefendorf, J. H. Dorroh, J. N. Gladding, C. H. Howell, H. C. Neuffer

1295

(6) **RICKARD, GROVER EDGAR,** Assoc. M., 108 North Twentieth St., Wheeling, W. Va. (Elected July 11, 1927.) (Age 41. Born Howe's Cave, N. Y.) 1913 A. B., and 1916 C. E., Cornell Univ. *TT 4: P 4.* April to Aug. 1916 Rodman and Inspector, City of Oneonta, N. Y. *TT 0.2: SP 0.2.*—Aug. to Sept. 1916 Sanitary Inspector, New York State Dept. of Health, inspecting sanitary conditions at private dwellings and municipalities on Long Island. *TT 0.1: SP 0.1.*—Oct. 1916-Nov. 1917 Draftsman on sewage-treatment works; Chemist in experimental sewage station; designed and was Asst. to Res. Engr. on construction of East 140th St. overflow channel. *TT 0.7: SP 0.5: P 0.2: D 0.2.*—Dec. 1917 to Feb. 1920 designed sewage-treatment works for Meadville, Pa., and water-treatment plant at Leetsdale, Pa.; Chemist and Bacteriologist; on preliminary design of sewage-treatment works for Erie, Pa.; supervised construction of water purification plants and operation of water plants, etc. *TT 1.8: SP 0.3: P 1.5: D 1.5.*—Feb. to Aug. 1920 with American City Eng. Co., had charge of Sewage-Treatment and Sewerage Dept., designed sanitary and storm-water sewers for Jeanette, Pa., Kent, Ohio, Studebaker Co., South Bend, Ind. and several sewage-treatment works. *TT 0.5: P 0.5: RC 0.5: D 0.5.*—Aug. 1920 to June 1923 Chemist and Bacteriologist, Water Dept., City of Memphis, Tenn., designed water mains for subdivisions and an iron-removal plant for a small town, supervised operation of swimming pools. *TT 2.8: P 2.8: D 0.5.*—June to Dec. 1923 supervised operation of swimming pools and designed 300 000-gal. iron removal plant for West Memphis, Ark. *TT 0.5: P 0.5: RC 0.5: D 0.2.*—Dec. 1923 to Dec. 1924 designed improvements to water plant at Shreveport, La. and sewage-treatment plant at Commodore, Pa.; Res. Engr. on construction at Commodore; supervised operation of water-treatment plants and collected chemical and bacteriological data for preliminary reports. *TT 0.9: P 0.9: D 0.6.*—Dec. 1924 to date with Wheeling (W.Va.) Water Works as Chemist and Supt. of Filtration of 20 000 000-gal. plant, in complete charge of operating the plant and since Aug. 1929 in charge of Eng. Dept. of water-works, including mapping and design of improvements to structures. *TT 7: P 7: RC 7: D 2.3.*—*TT 18.5: SP 1.1: P 17.4: RC 8: D 5.8.* Refers to H. L. Arbenz, J. T. Campbell, J. N. Chester, D. E. Davis, W. Donaldson, G. B. Gascoigne, W. L. Havens.

1296

(15) **WOODYARD JACOB FRANK, Jr.,** Assoc. M., Monterrey, Mex. (Elected June 3, 1915.) (Age 56. Born near Parkersburg, W. Va.) June 1893 to March 1897 West Point Cadet.—Winter 1897-1898 student, Univ. of Wis.—1898 1st Lieut. Volunteers. 1899 selling and installing machinery.—1900 to 1901 Draftsman successively for East Berlin (Conn.) plant, American Bridge Co. and for Brown Ketcham Iron Works, Indianapolis. *TT 0.5: SP 0.5.*—1901 to 1903 Draftsman and Checker, structural steel, Noelke-Richards Iron Works, Indianapolis. *TT 0.7: SP 0.7.*—1903 to 1906 Chf. Engr., The Denver (Colo.) Iron & Wire Works Co. *TT 3: P 3: RC 3: D 3.*—1906 to 1907 Asst. Chf. Engr. and later Chf. Engr., Westlake Constr. Co., St. Louis. *TT 1.5: P 1.5: RC 1.5: D 1.5.*—1907 Engr., Mauran Russell & Garden, Achts., St. Louis, New National Bank of Commerce and other buildings. *TT 0.5: P 0.5: RC 0.5: D 0.5.*—1907 to 1908 Engr. for Alfred Giles, Archt., Monterrey, Mex., constructing arch bridge, market house and various other reinforced concrete structures. *TT 1.3: P 1.3: RC 1.3: D 0.6.*—1909 to 1915 Archt., Engr. and Gen. Contr., on reservoir (500 000 gal.) and various reinforced concrete buildings, including office buildings, flour mills, warehouses, grain elevators, power and storage houses in Texas and Mexico, (names and location given in application) oil storage concrete reservoir and pump house, Monterrey steel plant, engineering design for 50 000-bu. storage bins, Tamalina Milling Co., San Antonio, etc., also many small jobs such as tanks, aqueducts, residences, towers, etc. *TT 6: P 6: RC 6: D 6.*—1915 to 1920 Gen Contr. and Cons. Engr., El Paso, Tex., built 5-story Lanier Bldg., El Paso High School Stadium and rebuilt El Paso Gas Works, Ft. Bliss Officers Club, water-supply and sewer extensions, and various small jobs. *TT 5: P 5: RC 4: D 1.*—1920 to 1922 various reinforced concrete work in Tampico, Mex. *TT 1.6: P 1.6: RC 1.6: D 1.*—1922 to date member of firm, J. F. Woodyard, Jr. & Son, architectural engineering and general contracting, Monterrey, Mex, designed and constructed many buildings in northern Mexico, principally reinforced concrete, including office buildings, warehouses, flour mill and grain elevators (names given in application); constructed cement plant and cement storage for Monterrey Cement Co.; power plant for Monterrey Light & Power Co., bulk stations, warehouses, filling stations, etc., for Texas Co. and Aguila Petroleum Co., hotel, residences, etc.

(given in detail in application). *TT 9.6: P 9.6: RC 9.6: D 9.6.*—*TT 29.7: SP 1.2: P 28.5: RC 27.5: D 23.2.* Refers to J. H. Brillhart, H. S. Crocker, A. T. Dusenbury, J. M. Howe, T. E. Huffman, W. H. Insley, D. Lee, J. A. McKim, J. T. L. McNew, H. C. Poske, C. L. Tindall, A. G. Trost.

FROM THE GRADE OF JUNIOR

1297

(1) **BARTON, GEORGE HENRY,** Jun., Apartado 683, Caracas, Venezuela. (Elected Jan. 16, 1928.) (Age 27. Born Natchez, Miss.) Student in Civ. Engr., Univ. of Ala. (Sept. 1923 to June 1924) and Rose Pol. Inst. (Sept. 1924 to June 1925). *TT 1: P 1.*—May 1922 to Sept. 1923 Clerk, Student Draftsman and Chf. Clerk, Cuyamel Fruit Company, in office of Chf. Engr., on railroads, drainage, irrigation, etc. *TT 0.7: SP 0.7.*—June to Sept. 1924 Instructor, Chimney Rock Camp for Boys.—June 1925 to Aug. 1926 Transitman, Asst. Engr. and Res. Engr., Carr & McFadden, Inc., West Palm Beach, Fla., on municipal layout, roads and streets, surveys, supervision of construction, etc. *TT 1.1: SP 0.1: P 1: RC 1: D 0.5.*—Aug. 1926 to Aug. 1927 Cost Engr., Ulen & Company, New York City, on computations, estimates, analysis reports, etc. *TT 1: P 1: RC 1.*—Sept. to Dec. 1927 Asst. to Supt., Henry Steers, Inc., New York City, progress reports, estimates, payrolls, etc. *TT 0.2: P 0.2.*—Dec. 1927 to July 1929 Asst. Engr. and Field Boss in location party, R. W. Hebard & Co., Inc., New York City, on highway (113 km.) and railroad (47 km.) location surveys in Republic of Colombia. *TT 1.7: P 1.7: RC 1.*—Aug. to Oct. 1929 Res. Engr. with F. E. Devlin, Cons. Engr., Wichita, Kans., on construction of city streets. *TT 0.2: P 0.2: RC 0.2.*—Oct. to Dec. 1929 Engr., Standard Dredging Co., New York City, dredging work on St. Mary's River, soundings, estimates, etc. *TT 0.2: P 0.2.*—Jan. 1930 to date Asst. Engr., Apure Venezuela Petroleum Corporation (Venezuelan Petroleum Co.), in charge of section road construction, allied works, etc. and since Dec. 1930 Dist. Supervisor, in general charge of road construction, transportation units, repair shops and Dist. Office. *TT 1.9: P 1.9: RC 1.9.*—*TT 8: SP 0.8: P 7.2: RC 5.1: D 0.5.* Refers to H. R. Faison, G. H. Hepburn, E. H. Jones. G. McFadden, P. H. Myers, G. W. Sackett, E. J. Shaw.

1298

(7) **CLARK, KENNETH MILES,** Jun., 3645 Sixteenth Ave. South, Minneapolis, Minn. (Elected Jan. 14, 1929.) (Age 28. Born Redwood Falls, Minn.) 1927 B. S. in C. E., Univ. of Minn. *TT 4: P 4.*—June to Aug. 1927 Computer, Minnesota State Highway Dept., St. Paul, on design of pavements, computing earthwork, estimating, etc. *TT 0.2: P 0.2: D 0.1.*—Aug. 1927 to March 1928 Asst. on Engr. Corps, Pennsylvania R. R., Chicago, Ill., instrument work, laying out yard, drafting, estimating, office work and construction. *TT 0.6: P 0.6: RC 0.1.*—April to Aug. 1928 Jun. Engr. on construction, San Dist. of Chicago, acting as Instrumentman and Jun. Engr. on North Side Contr. No. 10, intercepting sewer. *TT 0.4: P 0.4: RC 0.4.*—Aug. 1928 to Nov. 1929 Jun. Engr., U. S. Engrs., St. Paul, Minn., office work on flowage survey in connection with dam in Mississippi at Hastings, Minn., estimating, design. *TT 1.3: P 1.3: RC 0.5: D 0.5.*—Nov. 1929 to Jan. 1930 Asst. Engr., Bridge Dept., Great Northern Ry., St. Paul, Minn., on design of grain terminal. *TT 0.1: P 0.1: D 0.1*—Jan. 1930 to date Asst. Engr., Metropolitan Drainage Comm., St. Paul, Minn., on foundation study, soil analysis, hydraulic and reinforced concrete design. *TT 1.9: P 1.9: RC 1.9: D 1.9.*—*TT 8.5: P 8.5: RC 2.9: D 2.6.* Refers to F. Bass, J. A. Childs, A. S. Cutler, H. M. Hill, O. M. Leland, H. G. McNamee, J. I. Parcel.

1299

(13) **HARMAN, GEORGE SAMUEL** Jun., 119 Nineteenth Ave., San Francisco, Cal. (Elected July 12, 1926.) (Age 32. Born in Santa Cruz County, Cal.) Registered Civ. Engr., State of California— 1925 A. B. in C. E., Stanford Univ. *TT 4: P 4.*—June to Dec. 1925 Inspector with Chf. of Municipal Improvements, Longview, Wash., inspecting concrete for streets and sidewalks, etc. *TT 0.2: SP 0.2.*—June to Aug. 1926 Inspector with Director of Pacific Eng. Laboratories, inspected aggregate at bunkers for highway job. *TT 0.1: SP 0.1.*—Aug. 1926 to June 1927 Asst. to Research Mgr., Foster & Kleiser, San Francisco, Cal., made analytical cost data reports and charts from monthly financial reports. *TT 0.4: SP 0.4.*—June 1927 to Feb. 1928 Engr., Pacific Eng. Laboratories, supervised construction of concrete street improvements. *TT 0.7: SP 0.1: P 0.6: RC 0.6.*— May 1928 to date Asst. Engr., Playground Comm. of San Francisco, in charge of civil engineering. *TT 3: SP 0.5: P 2.5: RC 2.5: D 0.8.*—*TT 8.4: SP 1.3: P 7.1: RC 3.1: D 0.8.* Refers to R. S. Anderson, M. J. Callaghan, H. H. Ferrebee, C. C. Kennedy, L. A. Perry, E. N. Prouty.

1300

(13) **KERR, CLARENCE MARION**, Jun., 920 North First St., San Jose, Cal. (Elected Oct. 14, 1930.) (Age 31. Born Aurora, Nebr.) 1927 B. Sc. in C. E., Univ. of Nebr. *TT 4: P 4.*—Jan. to June 1922 Draftsman, Concrete Eng. Co., Omaha, Nebr. *TT 0.2: SP 0.2.*—Sept. 1925 to June 1926 Draftsman, Southern Pacific R. R. Co., San Francisco, Cal. *TT 0.4: SP 0.4.*—June 1926 to March 1928 with Santa Fe R. R., San Francisco, until Jan. 1927 as Draftsman, then Transitman, and after March 1927 Designer. *TT 1.3: SP 0.4: P 0.9: D 0.9.*—March 1928 to date Dist. Supt. of Constr., Granite Constr. Co., San Jose, Cal. *TT 3.7: P 3.7: RC 3.7.*—*TT 9.6: SP 1: P 8.6: RC 3.7: D 0.9.* Refers to E. O. Billwiller, C. B. Goodwin, S. P. Laverty, C. E. Mickey, G. L. Sullivan.

1301

(3) **KOFFSKY, SAMUEL**, Jun., 623 Myrtle Ave., Albany, N. Y. (Elected April 23, 1928.) (Age 29. Born Kiev, Russia.) 1923 C. E., Rens. Pol. Inst. *TT 4: P 4.*—June 1923 to Oct. 1924 Jun. Asst. Engr., Bureau of Highways, New York State Dept. of Public Works, drawing plans, computing earthwork and acting as Instrumentman on surveys, Inspector on construction and Supervisor of central concrete-mixing plant. *TT 0.8: SP 0.5: P 0.3: RC 0.3.*—Oct. 1924 to date Asst. Engr. with William Russell Davis, Cons. Engr., Albany, N. Y., detailing and designing superstructures and substructures for fixed and movable steel and concrete bridges, structural designs for buildings in collaboration with architects, preparing shop drawings for steel structures for Eastern Bridge & Structural Co. of Worcester, Mass., and for Union Structural Co. of Syracuse, N. Y., investigating safety of and detailing modifications in highway bridges and estimating and preparing bids for construction contracts; work included details for reconstruction of Hawk St. Viaduct at Albany (1925), details and design for large part of Peace Bridge at Buffalo, N. Y., and Ft. Erie, Ont., Canada (1926-1927) and for Michigan Ave. vertical lift bridge at Buffalo (1928-1929 not built, and 1931 new project), structural design for Arbor Hill Junior High School at Albany and structural design for a number of school and public buildings for J. Russell White, Archt., Albany (total contracts over $6 500 000). *TT 6.5: SP 0.6: P 5.9: RC 2.9: D 1.5.*—*TT 11.3: SP 1.1: P 10.2: RC 3.2: D 1.5.* Refers to A. G. Chapman, W. R. Davis, A. J. Dillenbeck, C. W. Dunham, J. M. Myers, C. W. Post, W. M. Stieve.

1302

(1) **LYONS, WILLIAM THEODORE**, Jun., 3671 Broadway, New York City. (Elected June 10, 1929.) (Age 29. Born New York City.) 1922 B. S., and 1924 B. S. in M. E., Coll. of City of N. Y. *TT 4: P 4.*—July to Sept. 1924 Transitman, Title Guarantee & Trust Co., New York City.—Sept. 1924 to Feb. 1926 Timekeeper, Transitman and Estimator, Geo. Colon & Co., Contrs., New York City, on hotels, apartments, theatres, lofts, etc. *TT 0.7: SP 0.7.*—Feb. 1926 to Feb. 1927 Supt. of Constr., John K. Turton & Co., Bldrs., New York City, on 15-story apartment structure. *TT 1: P 1: RC 0.8.*—Feb. to Nov. 1927 Archt.'s Supt. and Representative with Thos. W. Lamb, Archt., New York City, supervising construction of two theatres and a 15-story apartment. *TT 0.8: P 0.8: RC 0.8.*—Nov. 1927 to Oct. 1928 Chf. Engr., R. W. Hebard & Co., New York City, being Asst. to Supt. of Constr. on Panama City (Panama) branch ($300 000) of National City Bank of New York. *TT 1: P 1: RC 1.*—Nov. 1928 to March 1931 Supt. of Constr., Delta Constr. Co., New York City, in charge of construction of churches, garage, etc. *TT 2.3: P 2.3: RC 2.3.*—April 1931 to date Gen. Contr., residence construction in Pennsylvania, New York City and New Rochelle. *TT 0.7: P 0.7: RC 0.7.*—*TT 10.5: SP 0.7: P 9.8: RC 5.6.* Refers to A. Bartoccini, L. Fleischmann, H. B. Gates, R. W. Hebard, C. MacCallum.

1303

(10) **McGEHEE, CHARLES BURNAM**, Jun., 610 Rhodes-Haverty Bldg., Atlanta, Ga. (Elected June 7, 1926.) (Age 27. Born Bowling Green, Ky.) 1925 B. S. in C. E., Ga. School Tech. *TT 4: P 4.*—Jan. to Sept. 1924 with County Engr., Sutton County, Tex., as Instrumentman, Draftsman and Inspector on Federal Aid road projects. *TT 0.3: SP 0.3.*—July 1925 to Sept. 1927 Draftsman, Designer, Estimator and Chf. Engr. with Robert S. Fiske, Cons. Engr., Atlanta, Ga. *TT 1.9: SP 0.2: P 1.7: RC 1.7: D 1.9.*—Oct. 1927 to date with Truscon Steel Co., until April 1928 as Draftsman and Designer, Eng. Dept., Youngstown, Ohio, then Chf. Engr. and Eng. Salesman in Atlanta, Ga. office, and since Sept. 1930 Dist. Mgr. Atlanta branch. *TT 4.1: P 4.1: RC 3.5: D 2.*—*TT 10.3: SP 0.5: P 9.8: RC 5.2: D 3.9.* Refers to W. C. Caye, Jr., R. S. Fiske, O. W. Irwin, M. T. Singleton, F. C. Snow, W. C. Spiker.

1304

(9) MITCHELL, DESSO TWIGG, Jun., 1460 Republic Ave., Columbus, Ohio. (Elected Nov. 14, 1927.) (Age 31. Born Cloverdale, Ohio.) 1924 B. C. E., Ohio State Univ. *TT 4: P 4.*—June 1924 to Feb. 1926 with Pennsylvania Dept. of Highways, until June 1925 as Instrumentman, Erie and Kittanning Dists., on surveys, then Chf. of Survey Parties, Kittanning and Renovo Dists., in charge of and directing work of party. *TT 1.1: SP 0.4: P 0.7: RC 0.7.*—Feb. 1926 to date with Sewer Dept., Div. of Eng. and Constr., City of Columbus, Ohio, until June 1926 as Draftsman, assisting on contract drawings for sewer work, June 1926 to Jan. 1928 Asst. Engr., making computations for and on design of sewers and sewerage systems and preparing contract drawings, Jan. 1928 to Jan. 1929 Senior Asst. Engr., Jan. 1929 to Feb. 1930 Designing Engr., and since Feb. 1930 Field Engr., in charge of construction, directing inspection of methods and materials on sewer construction. *TT 5.5: SP 0.2: P 5.3: RC 4.4: D 0.9.*—*TT 10.6: SP 0.6: P 10: RC 5.1: D 0.9.* Refers to R. A. Allton, O. Bonney, E. G. Bradbury, J. H. Gregory, P. M. Holmes, J. T. More, C. E. Sherman, R. H. Simpson.

1305

(16) SLASON, EARLE BERNICE, Jun., 417 Grand Ave., Kansas City, Mo. (Elected Dec. 16, 1929.) (Age 31. Born Stockton, Kans.) Student, Kansas State Agri. Coll. (1917 to 1919) and Kansas State Univ. (1919 to 1922). *TT 2: P 2.*—July 1922 to Sept. 1925 Res. Engr., E. T. Archer & Co., Kansas City, Mo., in charge of construction. *TT 3.3: P 3.3: RC 3: D 0.3.*—Nov. 1925 to May 1927 Asst. to Chf. Engr., Union Bridge & Constr. Co., Kansas City, on substructure contract, Florida East Coast Ry. Co. *TT 1.5: P 1.5: RC 1.5.*—July 1927 to date with Constr. Materials Dept., American Steel & Wire Co., Chicago, Ill., on design of floor slabs, concrete pavement, reinforced concrete pipe, etc., assisting Consulting Engineers in improving the uses of wire-mesh reinforcing fabric, also sales and promotional work. *TT 4.3: P 4.3: RC 4.3: D 1.*—*TT 11.1: P 11.1: RC 8.8: D 1.3.* Refers to H. Allen, O. W. J. Anschuetz, W. H. Bosler, E. K. Carter, A. F. Reichmann, J. W. Shikles, H. A. Van Orman, E. C. L. Wagner, O. A. Zimmerman.

1306

(6) TURNER, WILL LAWRENCE, Jun., 302 South Third St., Martins Ferry, Ohio. (Elected June 8, 1930.) (Age 31. Born Martins Ferry, Ohio.) Sept. 1919 to June 1921 student, Coll. of Eng., Ohio Northern Univ. *TT 1: P 1.*—July 1921 to Jan. 1923 and June to Nov. 1923 Chainman and Transitman for Belmont County (Ohio) Surveyor. *TT 1: SP 1.*—Nov. 1923 to Aug. 1924 and Oct. 1924 to Sept. 1926 Engr., McElroy Eng. Co., Tampa, Fla., on drainage construction, surveys, locating and cross-sections of canals, responsible for progress and payment estimates, on various land and resident sub-division, and on design and construction of concrete structures, storm and sanitary sewer systems, streets, etc. *TT 2.7: P 2.7: RC 2.7.*—Aug. to Oct. 1924 Chainman, Instrumentman and Asst. Engr., on construction of I. C. H. No. 7, in Belmont County, Ohio, general field work on roadway and small structures, including data for payment estimates. *TT 0.2: P 0.2.*—Sept. 1926 to Sept. 1927 Res. Engr., Hillsborough County, Tampa, Fla., surveys, location, levels, cross-sections, design and construction of highways and roadway structures in North Tampa Road Dist. and part of East Tampa Dist., in responsible charge of field work and estimates. *TT 1: P 1: RC 1: D 1.*—Sept. 1927 to Jan. 1928 Constr. Engr., Ohio State Highway Dept., on over-head concrete and girder crossing, located on I. C. H. No. 7, in responsible charge of location, excavations and fills, driving bearing piles, erecting forms and retaining walls, placing of reinforcing steel, general field work and estimates. *TT 0.3: P 0.3: RC 0.3.*—Jan. 1928 to date City Engr., Martins Ferry, Ohio, on design and in responsible charge of all engineering projects, including estimating and involving sanitary- and storm-sewer systems, construction and reconstruction of city streets, specifications, investigations and examination of proposed water-supply systems, design and construction of water system for condensing purposes in Municipal Light & Power Plant, including erection of a concrete pump house 65 ft. below water level of Ohio River (work supervised over $500 000) *TT 3.9: P 3.9: RC 3.9: D 3.9.*—*TT 10.1: SP 1: P 9.1: RC 7.9: D 4.9.* Refers to H. L. Arbenz, G. P. Dunn, G. H. Elbin, F. A. Hastings, L. H. Lewis, P. F. Rossell.

1307

(1) WILLIG, WALTER LEE, Jun., 237 Passaic Ave., Hasbrouck Heights, N. J. (Elected Jan. 16, 1928.) (Age 29. Born Greenwich, Conn.) 1926 B. S. in C. E. and 1929 C. E., N. Y. Univ. *TT 4: P 4.*—July 1919 to Aug. 1923 (last year while student) with L. C. L. Smith, Cons. Engr., Long Island City, N. Y., as Rodman, Chainman, Transitman, Draftsman, Computer and Chf. of Party in charge of field work. *TT 1.6: SP 1.6.*—Aug. 1923

to Dec. 1930 (until June 1926 while student) with Robt. E. Carlin, Civ. Engr., Long Island City, N. Y., first as Computer and Chf. of Party, then Field Engr. on foundation, building and sewer construction and later Asst. in charge of computations, designs and drafting. *TT 4.4: P 4.4: RC 4.4: D 1.3.*——Dec. 1930 to date with New Jersey State Highway Comm., Jersey City, N. J., as Bridge Designer, checking shop detail drawings for heavy bridges and viaducts of Route 25 Connecting Link, also in charge of squad checking steel drawings on large steel contracts, involving approving or revising designs. *TT 0.9: P 0.9: RC 0.9: D 0.9.*——*TT 10.9: SP 1.6: P 9.3: RC 5.3: D 2.2.* Refers to W. Allan, J. Davidon, H. V. Disney, R. S. DuBois, S. Johannesson, D. Robertson, C. T. Schwarze, L. R. Shellenberger, S. A. Snook.

The Board of Direction will consider the applications in this list not less than thirty days after the date of its issue.

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AMERICAN SOCIETY OF CIVIL ENGINEERS

COMING MEETINGS

Board of Direction Meetings

January 18-19, 1932:

A Quarterly Meeting will be held at New York, N. Y.

Annual Meeting, New York, N. Y.

January 20, 21, and 22, 1932

January 20, 1932:

Morning. — Annual Meeting. Conferring of Honorary Membership, and Presentation of Medals and Prizes.

Afternoon. — Technical Meeting.

Evening. — President's and Honorary Members' Reception and Dinner Dance.

January 21, 1932:

Morning. — Technical Division Sessions.

Afternoon. — Technical Division Sessions.

Evening. — Entertainment and Smoker.

January 22, 1932:

All-Day Excursion.

The Reading Room of the Society is open from 9.00 A. M. to 5:00 P. M. every day, except Sundays and holidays; from May to September, inclusive, it is closed on Saturdays at 12:00 M.

Members, particularly those from out of town, are cordially invited to use this room on their visits to New York, to have their mail addressed there, and to utilize it as a place for meeting others. There is a file of 278 current periodicals, the latest technical books, and the room is well supplied with writing tables.

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AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

REPORTS

ANNUAL REPORT OF THE BOARD OF DIRECTION FOR THE YEAR ENDING DECEMBER 31, 1930

In compliance with the Constitution, the Board of Direction presents its Report for the year ending December 31, 1930.

THE SEVENTY-EIGHTH YEAR

In a résumé of a year of very great activity, the tendency to enumerate as many as possible of the accomplishments is strong. Each accomplishment and each new effort seem to warrant mention as a matter justifying enthusiasm for its own sake and as worthy of record if only for the record's sake. It will be the intent of this portion of this report, however, to expand somewhat upon only those matters which it now appears should be here recorded as significant in the life and structure of the Society and to indicate through specific mention of an item here or there the spirit which pervades the Society's work.

Changes in the Constitution

Three amendments to the Constitution prevailed in letter ballots during the year. By one amendment the number of meetings of the Board of Direction was set at a prescribed minimum of five instead of six, a matter deemed of convenience and economy only.

Another change in the Constitution voted upon favorably increased the number of Directors from 18 to 19, the intent being to provide a more equitable representation in the territory formerly included in District No. 11 consisting of Southern California and the Rocky Mountain States. The adoption of the amendment in turn required certain legal procedures which have been consummated. It also created a vacancy to which Roy C. Gowdy, M. Am. Soc. C. E., of Denver, Colo., was appointed. Mr. Gowdy's term will expire in January, 1932.

The Constitution was changed in a third manner at the suggestion of the members of the Board of Direction so as to increase the prescribed minimum requirements for the several grades of membership. In the statement prepared by the Executive Committee to inform the membership of the proposed change the thought was expressed that the written requirements were considerably below the interpretation placed upon them, and that it was but fair to all concerned that the provisions which were really operative should be more definitely expressed. A questionnaire forwarded to the membership at the direction of the Annual Meeting permitted a further clarification of the subject and the final vote of the membership on the amendments as subsequently changed was overwhelmingly in support. The new requirements became operative on October 31, 1930.

Functional Expansion Program

The adoption of the Functional Expansion Program by the Board of Direction at its January meeting accepted definitely for the Society active and comprehensive participation in the non-technical matters incident to the profession. New committees of an administrative nature and new committees to deal with matters of professional interest were authorized and appointed, each with a specific function. Three departments of Society work were established, Technical, Administrative, and Non-Technical. In effect there was devised and set in operation a system whereby many members of the profession especially interested and especially qualified were called upon to contribute of their interest and effort. Throughout the year these newly formed committees have been perfecting plans and in many instances building up a system of contact men through whom there is to be effected local application of approved procedures.

Of the non-technical committees three have brought forth practical results in specific form. The Committee on Registration of Engineers assumed vigorous leadership of the various engineering groups interested in the registration of professional engineers. Six prominent societies collaborated in producing a Uniform Registration Law which has been adopted and endorsed by eighteen independent engineering organizations. It stands to-day as the first widely accepted crystallization of practical thought upon the subject.

The Committee on Charges and Method of Making Charges for Professional Services has produced a report on that subject which was adopted by the Board of Direction, and distributed to the membership as Manual No. 5. The Committee has also produced for the use of clients an Epitome of the report, also adopted by the Board, and published as Manual No. 6. In this work the effort has been to acquaint the profession with established procedure in its details and ramifications and to inform clients of what may be expected in engineering services and what is due in recompense.

The Student Chapter Committee has sponsored the development of thirteen lantern lectures of important engineering works. Each lecture is prepared in mimeograph form and accompanied with from 40 to 70 slides. During the year these lantern lectures were made use of upon 160 occasions.

Civil Engineering

On October 1 appeared the first issue of *Civil Engineering,* a new monthly publication. *Civil Engineering* carries the more animated and graphic articles submitted for publication, and many Society announcements of official and semi-official character which formerly were contained in *Proceedings.* The *Proceedings* were thus simplified, making more readily accessible those contributions to engineering literature of the more studious nature. The change

contributes largely to a wider spread of knowledge of both Society and technical matters, providing, with *Proceedings,* two mediums different in style of expression and character of content.

Membership

Membership may be treated of in three different aspects—growth, interest, and activity. The net growth of Society Membership which has been marked throughout the past five years has continued. Notwithstanding the degree of unemployment which has been fully as observable among civil engineers as in general, the net growth in 1930 exceeds that of the previous year by 13 per cent.

The interest of the membership in Society affairs may perhaps be measured in part by attendance at Society meetings. The records show an attendance at the four meetings of the Society of 3 805, a larger figure than that of any preceding year. Perhaps, to an extent, this may be due to that feature of the Functional Expansion Program whereby the responsibility for the character and general management of each of the quarterly meetings of the Society is placed upon those resident in the region where the meeting is to be held.

The activity of the membership of the Society in behalf of its many programs and procedures is evidenced by the number participating. Of Society committees there are 208, with participating personnel of 1 219, and of Local Section Committees there are upward of 377, with participating personnel in excess of 1 530. It does not seem too much to say that, including those who contribute papers to the Society's publications and others who give individual efforts, not less than 3 350 members of the Society are engaged with greater or less intensity in the furtherance of the organized procedures.

Technical Developments

A detailed codification of the Society's various technical activities and the units through which they are effected was prepared and adopted by the Technical Procedure Committee. It sets forth with clarity the relations and functions of the Technical Divisions and their Committees and their work to the Special Committees of the Society, to the research work in process or contemplated, to the publications, and to the general administration of the Society. The Technical Divisions are the Society's specified agencies for the development of the technical matters other than those of a specific research nature. Upon them the Society depends for the orderly and progressive development of the art. Their production, together with that of the contributors to the publications and the accomplishments of the research committees, constitutes the contribution of the Society to the technical advance of the profession.

Summer School

The Society gave assistance to the Society for the Promotion of Engineering Education in a Summer School for Civil Engineering Teachers, held at Yale University, July 1 to July 23.

Accredited Schools

In view of the summary dismissal of many of the members of the faculties at the University of Mississippi and the Mississippi Agricultural and Mechanical College, and upon the recommendation of the Committee on Accredited Schools, it was voted that these Institutions be no longer considered Accredited Schools of Engineering within the significance of that term as set forth in the Constitution.

MEETINGS OF THE BOARD OF DIRECTION

There have been five meetings of the Board of Direction during 1930:

January 13-14, New York, N. Y.
January 16, New York, N. Y.
April 21-22, Sacramento, Calif.
July 7-8, Cleveland, Ohio
September 29-30, St. Louis, Mo.

There have been four meetings of the Executive Committee:

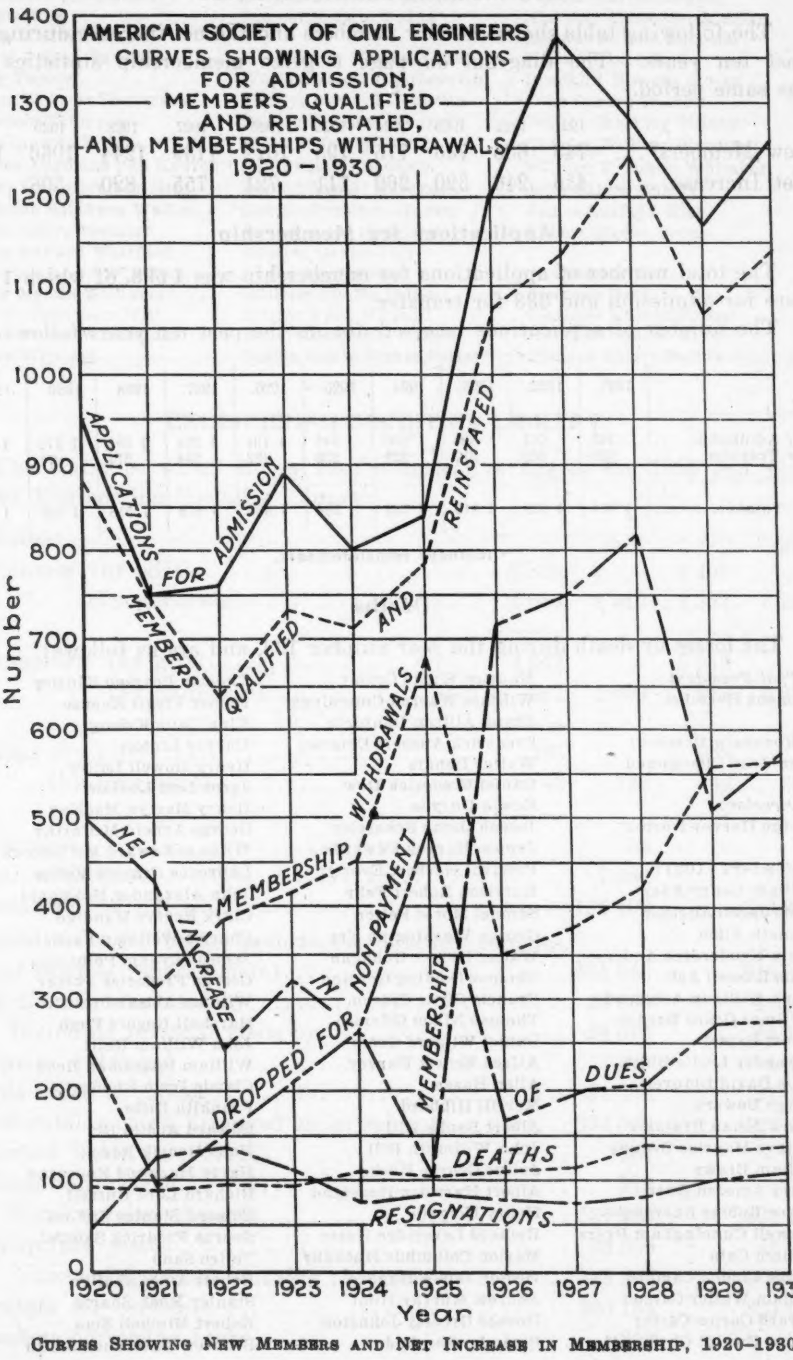
January 16, New York, N. Y.
April 21, Sacramento, Calif.
August 11, New York, N. Y.
December 15, New York, N. Y.

MEMBERSHIP

The changes in membership are shown in the following table:

	Jan. 1, 1930.			Jan. 1, 1931.			Losses:				Additions.			Totals.		
	Resident.	Non-Resident.	Total.	Resident.	Non-Resident.	Total.	Transfer.	Resignations.	Dropped.	Died.	Transfer.	Election.	Reinstatement.	Loss.	Gain.	Increase.
Honorary Members.	5	13	18	3	14	17	....	....	....	2	1*	.....	..	2	1	§1
Members	995	4 638	5 633	1 036	4 778	5 814	1	27	35	106	182†	156	12	169	350	181
Associate Members.	1 061	4 970	6 031	1 078	5 106	6 184	180	39	138	26	112‡	414	10	383	536	153
Juniors	468	1 785	2 253	545	1 957	2 502	112	36	145¶	...	.....	587	5	293	542	249
Affiliates	56	87	143	49	86	135	2	5	2	4	.....	5	..	13	5	§8
Fellows............	3	4	7	4	3	7	....	....	....	...	.....	.....	..	....	....	...
Total	2 588	11 497	14 085	2 715	11 944	14 659	295	107	320	138	295	1 112	27	860	1 434	574

* 1 Member.
† 180 Associate Members, 2 Affiliates.
‡ 112 Juniors.
§ Decrease.
¶ 47 Juniors dropped on account of age limit.



CURVES SHOWING NEW MEMBERS AND NET INCREASE IN MEMBERSHIP, 1920–1930.

New Members and Net Increase

The following table shows the new members and the net increase during the past ten years. The diagram on page 5 gives membership statistics for the same period.

	1921	1922	1923	1924	1925	1926	1927	1928	1929	1930
New Members*...	743	636	733	715	795	1072	1139	1244	1066	1139
Net Increase.....	435	246	320	262	111	721	755	820	508	574

Applications for Membership

The total number of applications for membership was 1 598, of which 1 260 were for admission and 338 for transfer.

The number of applications received during the past ten years follows:

	1921.	1922.	1923.	1924.	1925.	1926.	1927.	1928.	1929.	1930.
For Admission.....	745	761	896	800	848	1 194	1 374	1 284	1 172	1 260
For Transfer.......	309	260	217	239	256	292	304	274	271	338
Total...........	1 054	1 021	1 113	1 039	1 104	1 486	1 678	1 558	1 443	1 598

* Includes reinstatements.

Deaths

The losses by death during the year number 138, and are as follows:

Past-President:
Clemens Herschel

Honorary Member:
Léon-Jean Chaugnaud

Director:
George Harvey Norton

Members (105):
William Henry Adey
Louis Jacob Affelder
Kenneth Allen
Justin Kenderdine Anderson
Louis Russell Ash
Frank Milligan Ashmead
Mortimer Grant Barnes
Robert Bassel
Alexander Leslie Black
Louis David Blauvelt
George Bowers
Stoerk Johan Bratager
Gustave Maurice Braune
William Brown
Henry Amsden Burr
Hector Robins Burroughs
Maxwell Cunningham Byers
William Cain
George Linden Campen
William Waller Carson
Edward Carlos Carter
August Ewaldt Christhilf
Maurice Wurtz Cooley
William Warren Cummings
Frank Allison Danforth
Frederick Ausbert Delavau
Walter Dennis
Daniel Frederick Dow
Edwin Duryea
Joseph Oscar Eckersley
James Harvey Edwards
Percival Herbert Everett
Harrison Robert Fehr
Samuel Morse Felton
George Washington Fry
Walter Hamer Gahagan
Thomas Golding Gerdine
Francis Clyde Gideon
Thomas Nixon Gilmore
Daniel Wilgerd Gross
Alfred Ernest Harvey
Allen Hazen
Merrill Hibbard
Albert Banks Hill
John Willmuth Hill
Julius George Hocke
Albert Harrison Hogeland
Henry Holgate
Richard Leveridge Hoxie
Marion Columbus Huckaby
Hector James Hughes
Andrew Murray Hunt
Horace Greeley Johnston
Peter Junkersfeld
Thomas Pearson Kinsley
Homer Virgil Knouse
King Yang Kwong
Harvey Linton
Henry Howell Lotter
Jacob Lott Ludlow
Henry Martyn MacKay
George Arnold McCarthy
William Edward McClintock
Laurence Francis McCoy
John Alexander McDonald
Clark Rogers Mandigo
Charles Wellman Parks
James Suydam Polhemus
George Frederick Porter
William Abbott Pratt
Marshall Rogers Pugh
John William Raitt
William Boardman Reed
Claude Irvin Rhodes
Franklin Riffle
Edward Austin Rix
Hans Henrik Rode
Harry Harwood Rousseau
Richard Lord Russell
Edward Stanley Safford
George Fredrick Samuel
Tojiro Sano
Robert Ames Shailer
Stanley Rush Sharts
Robert Mitchell Sias
Bronson Hasbrouck Smith

***Members (Continued)*:**
Guy Sterling
Linton Waddell Stubbs
Hubert Primm Taussig
Harry Taylor
Arthur Webster Thompson
Benjamin Thompson
Louis Lincoln Tribus
Charles Augustus Van Keuren
Robert Lawrence Van Sant
Benjamin Southern Wathen
Albert Lowry Webster
James Edward Whitfield
Chauncey Grant Williams
Frank Martin Williams
Carlton Carpenter Witt
Irving Mason Wolverton
Wilkie Woodard
Morgan Edward Yeatman
Samuel McCain Young

***Associate Members* (26):**
William Saxon Charlesworth
Charles Edward Donnelly
Raymond Arden Edwards
William James Fulton
Arthur Bates Goodwin
Donald Ernest Harkness
Eugene Cochrane Harvey
Robert Grier Hemphill
Edward Baxter Hill
Edwin Allston Hill
William Chaffin Howe
Arthur Knox Mitchell
Robert McKnight Pardee
Waldo Justin Bickle Porter
Joseph Warren Rogers
Nicholas Alexander Scripko
Erastus Roland Simpson
Harold David Stoll
Franklin Stevens Storey
John Gerard Theban
Walter Checkley Tiffany
Charles William Wardle
William Franklin Whitaker
John Calvin Wilson
James Garfield Wise
Robert Elgene Yolton

***Affiliates* (4):**
Stedman Bent
Clarence Marvin Foster
Harry Wheelock King
Charles Albert Steinle

ENGINEERING SOCIETIES LIBRARY

The statistics which follow give comparative figures for 1929 and 1930 of the Engineering Societies Library:

		1929		1930
Additions:				
Volumes (by gift)	**2 826**		**2 400**	
" (by purchase)	**2 086**	**4 912**	**1 327**	**3 727**
Pamphlets (by gift)	**2 342**		**3 043**	
" (by purchase)	**357**	**2 699**	**214**	**3 257**
Maps (by gift)	**135**		**258**	
" (by purchase)	**43**	**178**	**43**	**301**
Searches		**95**		**75**
Total additions		**7 884**		**7 360**
Permanent collection		**135 408**		**139 916**
Expenditures for books, periodicals, binding, supplies, and salaries (approximate)		**$44 940**		**$48 000**
The Library was used by		**38 800**		**40 000**
Including personal visits by		**24 675**		**25 113**
Volumes cataloged		**5 289**		**4 276**
Cards added to catalog		**31 934**		**27 264**
Total catalog cards, arranged under subject		**394 079**		**421 343**
Searches made		**207**		**160**
Translations made		**215**		**246**
Total number of words		**587 500**		**691 700**
Photoprints made		**38 569**		**46 425**
Number of persons securing photoprints		**3 842**		**4 679**
Receipts for service		**$20 154**		**$21 560**
Members borrowing books		**313**		**250**

EMPLOYMENT SERVICE

The Employment Service has offices in New York, N. Y., Chicago, Ill., and San Francisco, Calif.

The number of men placed during 1930 has averaged about 89 per month. The following table shows the registrations and placements in the three offices:

Registrations and Placements in the New York, Chicago, and San Francisco Offices in 1930.

Month	Men Registered				Men Placed			
	New York	Chicago	San Francisco	Total	New York	Chicago	San Francisco	Total
January	102	65	159	326	63	10	26	99
February ...	96	76	113	283	55	11	28	94
March.......	101	75	157	335	60	11	36	107
April.......	127	68	126	321	61	15	45	121
May.........	113	85	132	330	51	14	30	95
June........	146	72	114	332	40	10	30	80
July........	144	84	115	343	51	12	22	85
August	122	58	133	313	40	20	29	89
September..	151	79	133	363	40	18	16	74
October.....	144	86	127	357	38	10	22	70
November ..	122	54	81	257	54	8	21	83
December...	107	62	96	265	48	7	16	71
Total......	1 475	864	1 486	3 825	598	146	321	1 068

PUBLICATIONS

The year 1930 has been marked by a radical change in the publications of the Society. A new publication, *Civil Engineering,* has been inaugurated, three numbers of which have been issued to the membership. In addition, ten numbers of *Proceedings,* Part I, and seven numbers of Part II; a volume of *Transactions;* a Year Book; Engineering Manuals Nos. 3 to 6, inclusive; and a new edition of "Aims and Activities", have been published.

Up to and including the September number, Part I of *Proceedings* consisted of Society Affairs, Papers, Reports, Discussions, and Memoirs, and Part II was the informal news section. With the appearance of *Civil Engineering* in October, the part devoted to Society Affairs was transferred thereto from *Proceedings.* In the future the latter will be devoted to the more technical papers, reports of Committees, and discussions; the Memoirs will appear in pamphlet form and in *Transactions.* Part II, the informal news section, was discontinued after the September issue.

The Engineering Manuals consist of the following: No. 3, "Lock Valve Practice", compiled by the Committee on Lock Valves of the Waterways Division of the Society; No. 4 "A Selected Bibliography on Construction Methods and Plant," compiled by the Executive Committee of the Construction Division; No. 5, "Charges and Method of Making Charges for Professional Services", the report of the Special Committee on Fees of the Society, J. Vipond Davies, *Chairman;* and No. 6, an Epitome of Manual No. 5 compiled by the Special Committee on Fees for the use of clients.

The stock of the various publications of the Society kept on hand for the convenience of members and others now amounts to 119 086 copies, the cost of which to the Society for paper and press work only has been $23 124.93.

The table (see page 10) shows the cost per page for text and illustrations in *Proceedings, Transactions,* and *Civil Engineering* for the past seventeen years.

The subjects of papers and discussions in *Proceedings, Transactions* and *Civil Engineering* during the year, and the number of pages devoted to each, follows:

Subject	*Transactions,* pages	*Proceedings,* pages	*Civil Engineering,** pages
City Planning	35	64	8
Contracts	90	10	....
Dams	68	292	12
Drainage and Irrigation	279	150	....
Engineering History	....	6	6
Engineering Profession	5	30	3
Engineering Research	....	....	3
Floods	65	....	6
Foundations	....	....	6
Handling of Materials	....	....	13
Highway Engineering	58	100	....
Hydrology, Hydraulics	107	242	....
Letters to the Editor	....	....	16
Materials of Engineering	22	134	10
Mathematics	25	84	....
Power Plants	52	22	....
Power Transmission	....	12	....
Railways	....	94	2
Rainfall	....	112	....
Sanitation	29	11	....
Sewage Disposal	132	92	24
Structural Engineering	223	212	29
Surveying	73	60	5
Transportation	....	82	10
Tunnels	....	46	....
Valuation	48	....	....
Water Power	....	16	....
Waterways	80	86	....
Water-Works	58	153	5
	1 449	2 060	158

*October to December, inclusive.

Summary of Publications for 1930

	Issues	Average edition	Total pages	Plates	Cuts
Proceedings, Part I (monthly numbers)	10	15 170	2 936	..	436
Proceedings, Part II (monthly numbers)	7	19 656	28	..	8
Civil Engineering (monthly numbers)	3	16 233	248	..	222
Transactions, Vol. 94	1	15 100	1 794	..	311
Year Book	1	14 800	528	1	3
Engineering Manual No. 3	1	2 000	104	..	37
" " No. 4	1	5 000	118	..	....
" " No. 5	1	15 300	56	..	2
" " No. 6	1	18 350	12	..	....
"Aims and Activities"	1	22 500	38	1	5
Total	27	....	5 960	2	1 024

TABLE SHOWING NUMBER AND COST OF PAGES AND COST OF ILLUSTRATIONS FOR *Transactions*, *Proceedings*, AND *Civil Engineering*.

Year.	*TRANSACTIONS.* Issues.	Edition.	Pages. Per volume.	Pages. Total.	*PROCEEDINGS, PART I.* Issues.	Edition.	Pages. Per volume.	Pages. Total.	*PROCEEDINGS* (PART I)* and *TRANSACTIONS.* Total pages.	Total cost.	Cost per page.	ILLUSTRATIONS. Cost.	Percentage of total cost.	Cost per page.
1914	1	8 200	1 968	16 140 000	10	8 150	4 076	33 220 000	49 360 000	$89 083.89	$0.00079	$2 963.32	7.6	$0.000075
1915	2	8 600	3 130	26 900 000	10	8 425	3 668	30 900 000	57 800 000	47 934.16	0.00083	3 684.68	7.7	0.000064
1916	1	8 400	2 301	19 330 000	10	8 350	2 892	24 140 000	43 470 000	35 645.65	0.00082	1 403.12	3.9	0.000032
1917	..	......	......	....	10	8 550	3 482	29 350 000	29 350 000	28 608.18	0.00097	3 708.97	12.9	0.000126
1918	1	8 700	1 879	16 340 000	10	8 950	2 341	20 950 000	37 290 000	33 785.64	0.00091	1 192.20	3.5	0.000032
1919	1	9 000	1 775	15 980 000	8	9 100	2 096	19 075 000	35 055 000	32 082.69	0.00091	1 128.53	3.5	0.000032
1920	..	......	......	....	10	10 142	2 014	20 440 000	20 440 000	23 446.34	0.00115	2 552.27	10.9	0.000125
1921	2	10 000 10 500	2 479 993	35 212 000	10	10 680	1 884	19 450 000	54 662 000	66 298.39	0.00121	2 034.72	3.1	0.000037
1922	1	10 200	1 826	19 900 000	10	11 100	2 740	30 400 000	50 300 000	56 200.00	0.00112	3 700.00	6.6	0.000073
1923	1	11 200	1 808	20 250 000	10	11 500	3 210	36 915 000	57 165 000	60 612.83	0.00106	4 809.88	7.9	0.000084
1924	1	11 500	1 515	17 440 000	10	11 750	2 612	30 700 000	48 140 000	47 573.72	0.00099	4 579.04	9.6	0.000095
1925	1	11 400	1 538	17 533 000	10	11 800	2 910	34 338 000	51 871 000	47 409.07	0.00091	6 359.19	13.4	0.000122
1926	1	12 100	1 786	21 610 000	10	12 200	3 046	37 161 000	58 771 000	53 473.54	0.00091	5 080.91	9.5	0.000087
1927	2	12 900 13 200	1 234 1 229	15 919 000 16 223 000	10	13 050	3 720	48 546 000	80 688 000	65 972.76	0.00080	9 279.49	14.0	0.000126
1928	1	14 000	1 860	26 040 000	10	14 200	4 016*	57 027 000	83 067 000	61 587.98	0.00074	7 249.00	12.4	0.000087
1929	1	14 500	2 030	29 435 000	10	14 540	3 566	51 850 000	81 285 000	58 388.39	0.00072	4 011.95	6.9	0.000049
1930	1	15 100	1 794	27 090 000	10	15 170	2 936	44 540 000	71 630 000	52 881.95	0.00074	4 189.56	7.9	0.000058
					CIVIL ENGINEERING									
1930	..	......	......	....	3	16 230	248	4 025 000	4 025 000	8 969.66	0.00223	995.06	10.1	0.000247

* Includes Part III, May 1928, 288 pp.

The cost of publications, as determined by the bills actually paid during the year, has been:

For Paper, Printing, etc., *Proceedings*, Part I	$28 591.28
For Paper, Printing, etc., *Proceedings*, Part II	2 268.62
For Paper, Printing, etc., of 15 643 Extra Copies of Papers, Reports, Discussions, and Memoirs	1 456.31
For Paper, Printing, Binding, etc., *Civil Engineering*	7 741.29
For 9 750 Extra Copies of Separate Papers for *Civil Engineering*	364.27
For Paper, Printing, etc., *Transactions* Vol. 94	10 844.18
For 5 525 Extra Copies of Separate Papers for *Transactions* Vol. 94	965.25
For Binding, Envelopes, etc., *Proceedings*, Part I	6 549.92
For Binding, *Proceedings*, Part II	534.02
For Envelopes, etc., *Civil Engineering*	209.94
For Binding,[1] Boxes, etc., *Transactions* Vol. 94	2 602.89
For Plates and Cuts, *Proceedings*, *Transactions*, etc.	4 189.56
For Plates and Cuts, *Civil Engineering*	995.06
For Development and Advertising Expenses, *Civil Engineering*	1 401.02
For Year Book	6 910.80
For Engineering Manual No. 3	640.96
For Engineering Manual No. 4	1 058.10
For Engineering Manual No. 5	1 159.19
For Engineering Manual No. 6	704.44
For Aims and Activities	589.76
For Copyright and Sundries, *Proceedings* and *Transactions*	104.10
For Copyright and Sundries, *Civil Engineering*	23.37
Total	$79 904.33
Deduct amount received from sale of publications	8 294.55
Net expenditures for publications in 1930	$71 609.78

READING ROOM OF THE SOCIETY

The attendance at the Reading Room during the year was 2 942.

Two hundred and eighty-three periodicals are regularly received. This number includes many foreign periodicals.

The list of recent engineering articles of interest which was prepared monthly by the assistants in the Reading Room, was discontinued with the September issue of *Proceedings*. To that date, the number of titles listed was 1 845 and they covered 63 pages. Fifty-six periodicals were indexed.

MEETINGS

Five meetings of 10 sessions were held during the year, as follows: At the Annual Meeting, at New York, N. Y., 1 (2 sessions); at the Spring Meeting

[1] Paper binding only.

at Sacramento, Calif., 1 (3 sessions); at the Annual Convention, at Cleveland, Ohio, 1 (2 sessions); at the Fall Meeting, at St. Louis, Mo., 1 (3 sessions); and 1 regular meeting of the Society held in the Engineering Societies Building, New York, N. Y.

At these meetings, there were presented 11 formal papers, many of which were illustrated with motion pictures and lantern slides, and 10 Addresses.

During the year 2 Progress Reports of Special Committees were published in *Proceedings,* both of which were presented at the Annual Meeting, as well as 22 papers that were not presented at any meeting of the Society. Digests of the Technical Papers and Discussions, and of reports of Division Committees presented at the Quarterly Meetings, were also published in *Proceedings* and in *Civil Engineering.*

The number of members and others who took part in the preparation and discussion of the papers, discussions, and reports of Committees published in *Proceedings* was 338. This does not include those who took part in the preparation and discussion of papers and reports of Committees presented at the meeting of the Technical Divisions, which meetings are listed on pages 15 to 17.

The total attendance at the five meetings of the Society was approximately 3 800. The registered attendance at the Annual Meeting was 1 747; at the Spring Meeting, 764; at the Annual Convention, 656; and at the Fall Meeting, 638.

The dates of the meetings of the Society during the year, together with the titles of papers (abstracts), addresses, reports of Special Committees, etc., presented thereat, are as follows:

January 15, 1930, Reports of the Special Committees on Engineering Employment in Public and Quasi-Public Offices,[2] and Irrigation Hydraulics.[3]

April 23, 1930 (Three Sessions)[4] "Some Reminiscences of Sacramento", by C. E. Grunsky, Past-President, Am. Soc. C. E.; "The California Plan for Conservation of Water Resources", by Edward Hyatt, M. Am. Soc. C. E.; and "State Supervision of the Design and Construction of Dams", by M. C. Hinderlider, M. Am. Soc. C. E.

July 9, 1930 (Two Sessions),[5] "Reflections on the Status of the Engineer",[6] Address by J. F. Coleman, President, Am. Soc. C. E.; "The Engineer's Professional Status", by William E. Wickenden, Esq.; and "The Shaker Heights Development, and Rapid Transit as a Part of the Terminal Plan", by William E. Pease, Assoc. M. Am. Soc. C. E.

October 1, 1930 (Three Sessions),[7] "Problems of a Large City": "Why the City", by W. F. Gephart, Esq.; "Municipal Preparedness", by E. R. Kinsey, M. Am. Soc. C. E.; "Supervised Regional Expansion", by A. P. Greensfelder, M. Am. Soc. C. E.; "Water Supply", by E. E. Wall, M. Am. Soc. C. E.; Municipal Drainage", by W. W. Horner, M. Am. Soc. C. E.; "Co-Ordination

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[2] *Proceedings,* Am. Soc. C. E., March, 1930, Papers and Discussions, p. 409.
[3] *Loc. cit.,* p. 589.
[4] *Loc. cit.,* August, 1930, Papers and Discussions, p. 1215 *et seq.*
[5] *Civil Engineering,* November, 1930, p. 71 *et seq.*
[6] *Transactions,* Am. Soc. C. E., Vol. 94 (1930), p. 1344.
[7] *Civil Engineering,* January, 1931, p. 234 *et seq.*

of Terminals", by F. G. Jonah, M. Am. Soc. C. E.; and "Mass Transportation", by R. J. Lockwood, M. Am. Soc. C. E.

The papers published in *Proceedings,* but not presented at any meeting of the Society, were as follows:

January, 1930, "Rainfall Characteristics and Their Relations to Soils and Run-Off"; by C. S. Jarvis, M. Am. Soc. C. E.; and "Plastic Flow in Concrete Arches", by Lorenz G. Straub, Jun. Am. Soc. C. E.

February, 1930, "Baldwin Filtration Plant, Cleveland, Ohio", by J. W. Ellms, G. W. Hamlin, A. G. Levy, and J. E. A. Linders, Members, Am. Soc. C. E.; "Laminated Arch Dams with Forked Abutments", by Fred A. Noetzli, M. Am. Soc. C. E.; and "The Chesapeake and Delaware Canal", by Earl I. Brown, M. Am. Soc. C. E.

April, 1930, "Completion of Moffat Tunnel of Colorado", by Clifford Allen Betts, M. Am Soc. C. E.; "Frequency and Intensity of Excessive Rainfalls at Boston, Massachusetts", by Charles W. Sherman, M. Am. Soc. C. E.; and "The Shannon Power Development in the Irish Free State", by A. R. C. Markl, Jun. Am. Soc. C. E.

May, 1930, "Analysis of Continuous Frames by Distributing Fixed-End Moments", by Hardy Cross, M. Am. Soc. C. E.; "The Training Wall Across the Liao Bar in Manchuria", by P. N. Fawcett, M. Am. Soc. C. E.; and "Evaporation as a Function of Insolation", by Burt Richardson, Esq.

August, 1930, "Highways as Elements of Transportation", by Fred Lavis, M. Am. Soc. C. E.

September, 1930, "Some Aspects of Water Conservation", by R. A. Sutherland, Assoc. M. Am. Soc. C. E.; "Construction of La Ola Pipe Line in Chile", by W. B. Saunders, M. Am. Soc. C. E.; "Tests of Broad-Crested Weirs", by James G. Woodburn, Assoc. M. Am. Soc. C. E.; and "Stresses in Gravity Dams by Principle of Least Work", by B. F. Jakobsen, M. Am. Soc. C. E.

October, 1930, "Theory of Similarity and Models", by Benjamin F. Groat, M. Am. Soc. C. E.; and "Formulas for Rainfall Intensities of Long Duration", by Merrill M. Bernard, M. Am. Soc. C. E.

November, 1930, "The Buttressed Dam of Uniform Strength", by Herman Schorer, Assoc. M. Am. Soc. C. E.; and "Economical Design of Hydraulic Pipe Line Dredge", by John Francis Cushing, M. Am. Soc. C. E.

December, 1930, "Flow of Water in Tidal Canals", by Earl I. Brown, M. Am. Soc. C. E.; and "The Don Martin Project", by Andrew Weiss, M. Am. Soc. C. E.

MEDALS, PRIZES, AND AWARDS

The award of Medals and Prizes for the year ending July, 1930, was as follows:

The Norman Medal to Charles Terzaghi, M. Am. Soc. C. E., for his paper entitled "The Science of Foundations—Its Present and Future".

The J. James R. Croes Medal to H. de B. Parsons, M. Am. Soc. C. E., for his paper entitled "Hydrostatic Uplift in Pervious Soils".

The Thomas Fitch Rowland Prize to R. McC. Beanfield, M. Am. Soc. C. E., for his paper entitled "Unusual Engineering Features of an Immense Theatre Building".

The James Laurie Prize to John H. Gregory, C. B. Hoover, and C. B. Cornell, Members, Am. Soc. C. E., for their paper entitled "The O'Shaughnessy Dam and Reservoir".

The Arthur M. Wellington Prize to George Gibbs, M. Am. Soc. C. E., for his paper entitled "The Virginia Railway Electrification".

The Phebe Hobson Fowler Professional Award: First Honor to Arthur W. Berresford, in recognition of his particularly efficient administration of American Engineering Council during the two years of his incumbency as its President. Second Honor to J. Vipond Davies, M. Am. Soc. C. E., in recognition of his accomplishment as Chairman of a Committee of the Society which developed the Report on Charges and Method of Making Charges for Professional Services, adopted by the Society.

The Phebe Hobson Fowler Award in Engineering Architecture: First Honor to Morris Goodkind, M. Am. Soc. C. E., for the design of the Raritan River Bridge, New Brunswick, N. J. Second Honor to Charles M. Spofford, M. Am. Soc. C. E., for the design of the Lake Champlain Bridge. Third Honor to G. F. Burch, M. Am. Soc. C. E., for the design of the Dixon Springs Bridge, Dixon Springs, Ill.

The Freeman Traveling Scolarship for the year 1930 was awarded to Hans Kramer, Assoc. M. Am. Soc. C. E.

LOCAL SECTIONS

There are at present 51 Local Sections, the Tacoma Section, approved by the Board January 13, 1930, having been added during the year.

The name of the Baltimore Section was changed to the Maryland Section, and that of the Western Washington Section to the Seattle Section.

TECHNICAL DIVISIONS

Each of the nine Technical Divisions held at least one session during the year. There were seventeen Division programs, of which four were double sessions, either all day or on two successive days, and one was a joint session between two Divisions.

Noticeable features of these Technical Sessions were excellent attendance and the fact that the members did not wander in and out of simultaneous sessions. Interest appeared to be sustained.

The policy of publishing abstracts of papers, discussions, and reports of committees presented at these sessions was continued during 1930. March and August *Proceedings*, and November, 1930, *Civil Engineering*, were the particular mediums for these abstracts.

City Planning Division

July 10, 1930,[8] "Cleveland Metropolitan Park System", by William A. Stinchcomb, M. Am. Soc. C. E.; Report on "Manual of City Planning, Scope and Procedure", by Hugh E. Young, M. Am. Soc. C. E.; and Report on "Manual on Subdivision of Urban Land", by Rolland S. Wallis, M. Am. Soc. C. E.

Construction Division

January 16, 1930,[9] "Hydraulic Plant and Its Application to Underpinning Structures Along the Nassau Street Subway", by Herbert M. Hale, M. Am. Soc. C. E.; "Plant Used on the Fulton Street-East River Subway", by Miles I. Killmer, M. Am. Soc. C. E.; and "Construction Methods and Plant on Cut-and-Cover Subway Work", by J. H. C. Gregg, Assoc. M. Am. Soc. C. E.

July 10, 1930[8] (Joint Session with Structural Division), "Design and Construction Features of the Goodyear-Zeppelin Airship Factory and Dock at Akron, Ohio", by Wilbur J. Watson, M. Am. Soc. C. E.; and "Design and Construction Features of the Cleveland Union Terminal", by C. P. Marsh, M. Am. Soc. C. E.

October 2, 1930,[10] "Twelve Month Construction", by James L. Taylor, Esq.; "Floating Foundations", by L. R. Viterbo, M. Am. Soc. C. E.; and "Caisson Foundations", by O. E. Mogensen and S. W. Bowen, Members, Am. Soc. C. E.

October 3, 1930,[10] "Pile Foundations", by A. C. Everham, M. Am. Soc. C. E.; and "Chicago Open Well Method of Foundations", by W. J. Newman, Esq.

Highway Division

January 15, 1930,[11] Report on the Status and Progress in the Art of Highway Engineering, by Henry B. Drowne, M. Am. Soc. C. E.

January 16, 1930,[12] "Highway Construction Management", by T. Warren Allen, M. Am. Soc. C. E.; "Construction Management", by Richard Hopkins, Esq.; and Résumé of the Papers and Discussions at the Quarterly Meetings during 1929 on the Subject, "Equitable Discussion for Highway Purposes of Motor Vehicle License Fees and Gasoline Taxes", by A. N. Johnson, M. Am. Soc. C. E.

April 24, 1930,[13] "Pre-Qualification of Contractors from the Standpoint of the Engineer", by C. H. Purcell, Assoc. M. Am. Soc. C. E.; "Pre-Qualification

[8] *Civil Engineering,* November, 1930.
[9] *Proceedings,* Am. Soc. C. E., March, 1930, Papers and Discussions, p. 531 *et seq.*
[10] *Civil Engineering,* January, 1931.
[11] *Proceedings,* Am. Soc. C. E., March, 1930, Papers and Discussions, p. 427.
[12] *Loc. cit.,* p. 465.
[13] *Loc. cit.,* August, 1930, Papers and Discussions, p. 1245.

of Contractors: Legal Aspects", by L. I. Hewes, M. Am. Soc. C. E., "Pre-Qualification of Contractors from the Standpoint of the Contractor", by Walter Wilkinson, Esq.; "Low-Cost Bituminous-Treated Crushed Rock and Gravel Roads", by Walter N. Frickstad, M. Am. Soc. C. E.; and "Western Highway Practice", by C. S. Pope, M. Am. Soc. C. E.

October 2, 1930,[13a] "Pre-Qualification of Contractors", by Robert B. Brooks, M. Am. Soc. C. E., Alan J. Parrish, Esq., Ward P. Christie, Assoc. M. Am. Soc. C. E., and W. W. Zass, M. Am. Soc. C. E.

October 3, 1930,[13a] "Designing of State Highway Systems", by T. H. Cutler, Esq.; and "County Highways", by Roy W. Jablonsky, Assoc. M. Am. Soc. C. E.

Irrigation Division

April 24, 1930,[14] "The Proposed Colorado Aqueduct and Metropolitan Water District", by Frank E. Weymouth, M. Am. Soc. C. E.; "Foundation Treatment of the Rodriguez Dam on the Tijuana River, Mexico", by Charles P. Williams, M. Am. Soc. C. E.; and "Hydro-Electric Power Development as an Aid to Irrigation", by C. C. Cragin, M. Am. Soc. C. E.

Power Division

January 15, 1930,[15] Report on the Status and Progress in the Art of Power Engineering, by George A. Orrok, M. Am. Soc. C. E.

January 16, 1930,[16] "State Supervision of the Design and Construction of Dams" by A. H. Markwart, M. Am. Soc. C. E.

Sanitary Engineering Division

January 15, 1930,[17] Report on the Status and Progress in the Art of Sanitary Engineering, by C. A. Emerson, Jr., M. Am. Soc. C. E.

January 16, 1930,[18] Reports of Committees on Municipal Cleansing Practice, Rudolph Hering Medal Award, Filtering Materials from Water and Sewage Works,[19] Friction of Sludge in Pipes, New Jersey Sewage Disposal Experiments, and Water-Works.

July 10, 1930,[20] Reports of Committees on Sewage Works Operation and Control, and Filtering Materials for Trickling Filters.

[13a] *Civil Engineering*, January, 1931.
[14] *Proceedings*, Am. Soc. C. E., August, 1930, Papers and Discussions, p. 1283.
[15] *Loc. cit.*, March, 1930, Papers and Discussions, p. 437.
[16] *Loc. cit.*, p. 473.
[17] *Loc. cit.*, p. 443.
[18] *Loc. cit.*, p. 517.
[19] *Loc. cit.*, September, 1930, Papers and Discussions, p. 1851.
[20] *Civil Engineering*, November, 1930.

Structural Division

January 16, 1930,[21] "Plans and Research—Kill van Kull Bridge", by O. H. Ammann, M. Am. Soc. C. E.; and "Design and Erection—Kill van Kull Bridge", by Allston Dana, M. Am. Soc. C. E.

April 24, 1930,[22] "Southern Pacific Company's Suisun Bay Bridge", by W. H. Kirkbride, M. Am. Soc. C. E.; and "Salt Springs Dam", by O. W. Peterson, M. Am. Soc. C. E.

Surveying and Mapping Division

January 15, 1930,[23] Report on the Status and Progress in the Art of Surveying and Mapping, by William Bowie, M. Am. Soc. C. E.

April 24, 1930,[24] "Preliminary Topographic Surveys for Proposed Colorado River Aqueduct", by E. A. Bayley, M. Am. Soc. C. E.; and "The Aerocartograph Method of Photo-Topographic Mapping", by C. H. Birdseye, M. Am. Soc. C. E.

October 3, 1930,[25] "Surveying and Mapping Activities in Connection with Flood Control Work of the Mississippi River", by L. D. Worsham, M. Am. Soc. C. E.; and "Aerial Survey—St. Louis-Osage Transmission Lines", by Messrs. Stanley Stokes and F. G. Dana.

Waterways Division

July 10, 1930,[26] "Coal and Car Dumper Plant of the New York Central Railroad at Toledo, Ohio", by B. R. Leffler, M. Am. Soc. C. E.

October 2, 1930,[27] "Brazos River Improvement", by Major Milo P. Fox, Corps of Engrs., U. S. A.; and "Terminals on Inland Waterways", by Walter F. Schulz, M. Am. Soc. C. E.

Membership of Technical Divisions

There are now 13 067 members enrolled in the Technical Divisions as follows:

City Planning Division	1 497
Construction Division	2 334
Highway Division	2 015
Irrigation Division	865
Power Division	759
Sanitary Engineering Division	1 558
Structural Division	2 533
Surveying and Mapping Division	732
Waterways Division	774

[21] *Proceedings*, Am. Soc. C. E., March, 1930, Papers and Discussions, p. 487.
[22] *Loc. cit.*, August, 1930, Papers and Discussions, p. 1307.
[23] *Loc. cit.*, March, 1930, Papers and Discussions, p. 453.
[24] *Loc. cit.*, August, 1930, Papers and Discussions, p. 1335.
[25] *Civil Engineering*, January, 1931.
[26] *Loc. cit.*, November, 1930.
[27] *Loc. cit.*, January, 1931.

STUDENT CHAPTERS

There are at present 95 Student Chapters. The Catholic University of America Student Chapter was organized during 1930.

The following Chapters were discontinued:

Ole Miss (University of Mississippi)
Mississippi Agricultural and Mechanical College

Lantern Lectures

In October, 1929, a new service to the members of Student Chapters was initiated, with the preparation and circulation of a series of lantern lectures on important engineering works. The material for these lectures was obtained from members of the Society who had been connected with the projects described. The service was started with eight different titles, and duplicate sets of slides were provided, so that sixteen lectures were in circulation from November, 1929, through May, 1930. During that time the lectures were used upon one hundred and twenty-six occasions.

In October, 1930, the number of different titles was increased to thirteen, so that, including the ones already prepared, there are now twenty-six sets of slides in use. From the beginning of the school year up to December 31, 1930, fifty-three lectures had been given, and many more are already reserved for the early part of 1931. During the year 1930 the total number of lectures delivered was 160. The groups using the lecture material include fifty-six Student Chapters of the Society, and about a dozen other organizations, such as Local Sections, and occasional non-member groups, where the lecture has been loaned to a member without interfering with requests from the Chapters.

FINANCES

A payment of $15 000 was made on the mortgage. There is cash on hand in the amount of $30 209.49, exclusive of the various Trust Funds, etc., which amount to $8 259.40. The sum of $25 233.07 has temporarily been invested in short-term securities.

The reports of the Secretary and Treasurer are appended.

By order of the Board of Direction,

GEORGE T. SEABURY, *Secretary.*

January 19, 1931.

REPORT OF THE TREASURER OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS FOR THE YEAR ENDING DECEMBER 31, 1930

In compliance with the provisions of the Constitution, I have the honor to present the following report:

Cash on hand January 1, 1930		$41 677.14
Receipts.		
From Current Sources, January 1 to December 31.	$320 622.31	
Rent from 57th Street Property	52 500.00	
From Engineering Foundation for:		
Special Committee on Concrete and Reinforced Concrete Arches	1 500.00	
Special Committee on Steel Column Research	2 000.00	
Special Committee on Earths and Foundations.	2 500.00	
The Freeman Fund Investments	1 664.50	
Interest on Temporary Investments	1 719.94	
Maturity of Various Securities	74 603.47	
Merritt H. Smith Memorial	1 103.34	458 213.56
Disbursements.		
Payment of Bills by Audited Vouchers for Current Business, January 1 to December 31	$391 180.42	
Payment on Mortgage, 57th Street Property	15 000.00	
Purchase of Securities	55 241.39	
Cash on hand, December 31, 1930	38 468.89	
	$499 890.70	$499 890.70

Respectfully submitted,

OTIS E. HOVEY,
Treasurer.

BALANCE SHEET

ACCOMPANYING THE REPORT

ASSETS

Real Estate:			
Interest in real estate and other assets of Engineering Foundation, Inc., exclusive of trust funds		$493 352.60	
220 West 57th St., New York, N. Y., at estimated value, less depreciation		617 001.24	$1 110 353.84
Equipment:			
Furniture and office equipment		$75 087.42	
Less, reserve for depreciation		66 002.86	9 084.56
Library:			
Cash expended for books, etc		$22 122.22	
Donations (estimated)		72 310.83	94 433.05
Investments in Chicago and Cook County tax notes (partly past due), at cost			25 233.07
Fund Investments:			
Library and Prize Funds, etc.:			
U. S. Liberty, New York City, and R. R. bonds, at cost	$38 684.75		
Uninvested cash	206.03	$38 890.78	
The Freeman Fund:			
Securities, at cost	$25 898.93		
Uninvested cash	34.57	25 933.50	
Phebe Hobson Fowler Fund:			
Securities, at nominal value		1.00	
The Fifty-seventh Street Property Fund:			
Securities, at cost	$49 079.24		
Uninvested cash	632.14		
Accrued interest on investments	609.96	50 321.34	
Alfred Noble Fund:			
Securities, at cost	$15 416.25		
Unexpended cash	52.24	15 468.49	130 615.11
Working Assets:			
Publications on hand		$23 124.93	
Unexpired insurance premiums		397.70	23 522.63
Current Assets:			
Cash:			
In banks, general		$25 012.80	
In hands of Secretary		5 000.00	
On deposit with U. S. Post Office		200.00	
In banks, for special purposes (*per contra*)		7 531.11	37 743.91
Due from members, in arrears			21 620.64
Due from non-members			1 099.33
Accrued interest on investments:			
On general investments		$2 676.79	
On specific fund investments (*per contra*)		252.50	2 929.29
			$1 456 635.43

We have examined the accounts of the AMERICAN SOCIETY OF CIVIL
of investments at cost and assuming that the estimate of the property and library
that, in our opinion, the above balance sheet sets forth correctly the financial

NEW YORK, January 12, 1931.

DECEMBER 31, 1930

OF THE SECRETARY

LIABILITIES AND FUNDS

Mortgage payable, due February 1, 1932		$45 000.00	
Accrued interest on mortgage		937.50	$45 937.50
1931 dues paid in advance			62 967.50
Unexpended balances of cash received for special purposes (*per contra*):			
Committee on Stresses in Railroad Track		$ 360.28	
Soils Committee		1 300.00	
Power Division		2 078.01	
City Planning Division		435.90	
Surveying and Mapping Division		55.00	
Rudolph Hering Medal Fund		497.11	
Freeman Fund income		707.70	
Phebe Hobson Fowler Fund income		130.52	
Alfred Noble Fund		863.25	
Merritt Haviland Smith Memorial Fund		1 103.34	7 531.11
Interest accrued but not yet received on funds invested in securities (*per contra*):			
Alfred Noble Fund			252.50
Funds:			
Herbert Stewart Library Fund	$ 2 000.00		
Joseph G. Swift Library Fund	1 000.00		
Compounded Dues Fund	15 480.00		
Norman Medal Fund	1 000.00		
Rowland Prize Fund	1 222.50		
Collingwood Prize Fund	1 000.00		
Arthur M. Wellington Prize Fund	2 150.00		
Fellowship Fund	13 038.28		
Hiram F. Mills Legacy	2 000.00	$38 890.78	
The Freeman Fund		25 933.50	
Phebe Hobson Fowler Fund		1.00	
The Fifty-seventh Street Property Fund		50 321.34	
Alfred Noble Fund		15 468.49	130 615.11
Surplus, including amount arising from revaluation of real estate			1 209 331.71
			$1 456 635.43

ENGINEERS for the year ended December 31, 1930, and (based upon the valuation valuations and book inventory of publications on hand are correct) we certify condition of the Society at that date.

LYBRAND, ROSS BROS. & MONTGOMERY,

Accountants and Auditors.

REPORT OF THE SECRETARY FOR THE

TO THE BOARD OF DIRECTION OF THE

GENTLEMEN:—I have the honor to present a statement of Receipts and There is also appended a general Balance Sheet showing the condition of the

RECEIPTS

Balance on hand January 1, 1930			$41 677.14*
Entrance Fees		$ 23 045.00	
Current Dues		180 006.59	
Past Dues		11 798.32	
Advance Dues		62 967.50	
Binding for Members		9 131.00	
Badges		6 359.75	
Certificates of Membership		879.50	
Sale of Publications		8 294.55	
Interest on Deposits		783.47	
Interest on Investments		3 350.19	
Interest Accrued		1 923.09	
Annual Meeting		5 089.57	
Postage		252.93	
Miscellaneous		609.54	
The Freeman Fund:			
Income and Expense		1 664.50	
Principal		1 403.54	
City Planning Division:			
1930 Dues		60.00	
Interest		30.90	
Surveying and Mapping Division:			
1930 Dues		15.00	
Power Division: Interest		157.28	
Maturity of Bonds		74 603.47	
Income from 57th St. Property:			
Credited to General Receipts	$41 000.00		
" " 57th St. Property Fund	11 500.00	52 500.00	
Interest on Invested Funds		1 685.46	
The Alfred Noble Fund:			
Interest on Invested Funds		817.59	
The Rudolph Hering Medal Fund:			
Interest		41.14	
The Merrit H. Smith Memorial:			
Principal		1 076.50	
Interest		26.84	
Advertising		3 640.34	
From Engineering Foundation in Credit to:			
Special Committee on Earths and Foundations	$ 2 500.00		
Special Committee on Steel Column Research	2 000.00		
Special Committee on Concrete and Reinforced Concrete Arches	1 500.00	6 000.00	458 213.56
			$499 890.70

* For itemized statement see p. 24.

YEAR ENDING DECEMBER 31, 1930

AMERICAN SOCIETY OF CIVIL ENGINEERS.

Disbursements for the fiscal year of the Society, ending December 31, 1930.

affairs of the Society.

Respectfully submitted,

GEORGE T. SEABURY,
Secretary.

DISBURSEMENTS

Salaries of Officers	$ 21 725.00	
Retirement Allowances	6 954.48	
Clerical Help	102 706.06	
Traveling Allowance of Officers	19 256.45	
Rent	12 901.98	
Telephone	2 034.11	
General Publications	10 614.54	
General Printing	4 564.91	
Postage	13 676.23	
Binding of *Transactions*	4 784.28	
Badges	4 682.23	
Certificates of Membership	546.96	
Annual Prizes	466.83	
Office Supplies	3 464.05	
Furniture—Office Equipment	4 328.88	
Current Business	5 272.37	
Interest on Mortgage	2 625.00	
Insurance	227.65	
Reading Room	507.90	
57th St. Property Fund:		
Purchase of Securities	12 990.00	
Employment Service	605.00	
Miscellaneous	1 162.02	
Library	9 185.64	
American Standards Association	532.00	
American Engineering Council	14 082.00	
Local Sections	11 426.00	
Summer School for Engineering Teachers	2 500.00	
Technical Publications	69 289.79	
Meetings	15 688.39	
Technical Divisions	8 683.75	
Technical Committees	13 460.10	
Administrative Committees	2 461.58	
Professional Committees	2 555.40	
Payment on Mortgage	15 000.00	
Purchase of Bonds	55 241.39	
Interest Accrued	1 653.82	
The Freeman Fund:		
Income and Expense	1 800.00	
Principal	1 244.42	
The Alfred Noble Fund:		
Refund of Contribution	500.00	
The Phebe Hobson Fowler Fund:		
Income and Expense	20.60	$461 421.81
Cash on hand December 31, 1930		38 468.89*
		$499 890.70

* For itemized statement see p. 24.

Report of Secretary (*Continued*)

Itemized Statement of Cash on Hand January 1, 1930

Society Funds in Chase National Bank, 23d Street..	$28 759.46	
" " " " " 41st Street..	500.00	
Petty Cash (in hands of Secretary).................	5 000.00	$34 259.46
Dues Collected for:		
Power Division, 1923–1928........................	$1 919.73	
City Planning Division, 1926–1929................	345.00	
Surveying and Mapping Division, 1928–1929........	50.00	2 314.73
For Special Committee on Stresses in Railroad Tracks...........		1 120.28
The Rudolph Hering Medal...........................		455.97
The Freeman Fund:		
Principal..	$0.45	
Income...	450.18	450.63
57th Street Property Fund..........................		436.68
Soils Committee (special account)..................		1 300.00
Alfred Noble Fund..................................		607.24
Phebe Hobson Fowler Fund...........................		151.12
In Escrow, E. W. L.................................		500.00
Library and Prize Fund.............................		81.03
		$41 677.14

Itemized Statement of Cash on Hand December 31, 1930

Society Funds in Chase National Bank, 23d Street..	$24 512.80	
" " " " " 41st Street..	500.00	
Petty Cash (in hands of Secretary).................	5 000.00	$30 012.80
57th Street Property Fund..........................		632.14
Power Division.....................................	$2 078.01	
City Planning Division.............................	435.90	
Surveying and Mapping Division.....................	55.00	2 568.91
Special Committee on Stresses in Railroad Tracks...	$360.28	
Special Committee on Earths and Foundations (special account).........	1 300.00	1 660.28
The Freeman Fund:		
Principal..	$34.57	
Income and Expense.................................	314.68	349.25
In Escrow, E. W. L.................................		393.02
The Alfred Noble Fund:		
Uninvested...	$52.24	
Not Released.......................................	55.00	
Income and Expense.................................	808.25	915.49
Phebe Hobson Fowler Fund...........................		130.52
Merritt H. Smith Memorial Fund.....................		1 103.34
Rudolph Hering Medal...............................		497.11
Library and Prize Fund.............................		206.03
		$38 468.89

Report of Tellers on Second Ballot for Official Nominees

"October 15, 1931.

"To the Secretary,
American Society of Civil Engineers:

"The Tellers appointed to canvass the Second Ballot for Official Nominees report as follows:

"Total number of ballots received		3 180
"Deduct:		
Ballots from members in arrears of dues	58	
Ballots not signed	22	
Ballots with printed signature	1	
Ballots from members who have died since voting	1	
"Total not entitled to vote		82
"Ballots canvassed		3 098

"For Vice-President, Zone I:

Arthur S. Tuttle	1 073
Blank	32
Void	2
Total	1 107

"For Vice-President, Zone IV:

D. C. Henny	744
Blank	6
Void	1
Total	751

"For Director, District 3:

E. P. Lupfer	220
Blank	6
Void	1
Total	227

"For Director, District 8:

M. L. Enger	213
Blank	2
Void	0
Total	215

"For Director, District 5:

John H. Gregory	204
Blank	2
Void	0
Total	206

"For Director, District 9:

Robert Hoffmann	227
Blank	1
Void	0
Total	228

"For Director, District 7:

H. E. Riggs	253
Blank	3
Void	0
Total	256

"For Director, District 12:

J. C. Stevens	163
Blank	6
Void	0
Total	169

"For Director, District 16:

E. B. Black	167
T. A. Leisen	70
R. C. Gowdy	98
Blank	0
Void	0
Total	335

"Respectfully submitted,
"W. L. Cadwallader, *Chairman*,

"John W. Daly,
Philip Sander,
Theodore P. Kilian,
Henry W. Troelsch,

"C. E. Sudler,
Roy G. Butler,
Hans R. Jacobsen,
W. H. Barton,

E. W. Bowden,

"Tellers."